HR-1042

PRESSURE RELIEF AND

OTHER JOINT REHABILITATION TECHNIQUES

FINAL REPORT

By

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Prepared For

U.S. Department Of Transportation
Federal Highway Administration
Office Of Enginnering and Highway
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Pavement Division
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16. Abstract

A study of four major concrete pavement joint rehabilitation techniques has been conducted, including: pressure relief joints, full-depth repairs, partial-depth repairs and joint resealing. The products of this research include the following for each technique: a summary of published research, detailed documentation of the design and performance of the 36 projects, conclusions and recommendations of the state highway engineers panel, "Design and Construction Guidelines" and "Guide Specifications." The latter two products are prepared for use by state highway agencies.

The results of this study are based upon a review of literature, extensive field surveys and analysis of 36 rehabilitation projects, and the experience of an expert panel of state highway engineers.

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One of the most beneficial and helpful aspects of this study was the panel of eight expert state highway engineers. The panel members spent many hours reviewing documents and in meetings providing guidance to the research staff. They also assisted in the collection of the rehabilitation project data. Much appreciation is extended to the members of the expert panel:

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CHAPTER I

INTRODUCTION

Problem Definition

One of the most common and difficult rehabilitation problems with jointed concrete pavements is the repair, replacement and restoration of transverse joints. These joints were originally designed to accommodate changes in slab length caused by temperature and drying shrinkage effects. However, over time and with traffic, the effectiveness and capability of these joints is often reduced by sealant deterioration, intrusion of incompressibles, concrete deterioration, construction errors, corrosion of load transfer devices, loss of support of slab corners and loss of load transfer ability due to repeated heavy traffic loadings.

Most of these problems (with the exception of faulting and corner breaks) are associated with long-jointed reinforced concrete pavements (JRCP), and not with short-jointed plain concrete pavements (JPCP).

The resulting effect of all these factors is the deterioration of the joint through spalling, corner breaks, faulting and blowups. If the joint locks up because of corrosion, it will even contribute to the opening of transverse cracks, causing them to spall and fault.

In response to extensive joint deterioration, many agencies have developed joint repair techniques largely through trial and error. These include full-depth repairs, partial-depth repairs, cleaning and resealing of joints, pressure relief joints, and load transfer restoration. The success of these procedures has often been poor with very short service lives. Thus, there is a great need for improved procedures for repair and restoration of joints.

The occurrence of blowups is the most serious joint deterioration problem in terms of potential danger to the public and expense and difficulty of repair (see Figure 1). Several research studies have been conducted to develop pressure relief joint usage, design, and construction criteria. Many agencies have installed pressure relief joints on long-jointed JRCP. The performance of these joints and their effects on adjacent pavements has been less than satisfactory.

Objectives

The objectives of this project are as follows:

1. Identify, define, and document the criteria for using pressure relief joints to relieve and/or reduce the rate of deterioration of existing joints in concrete pavements.

2. Identify, update, and document the current technology of other joint rehabilitation techniques for the repair, replacement, or restoration of existing joints in concrete pavements.

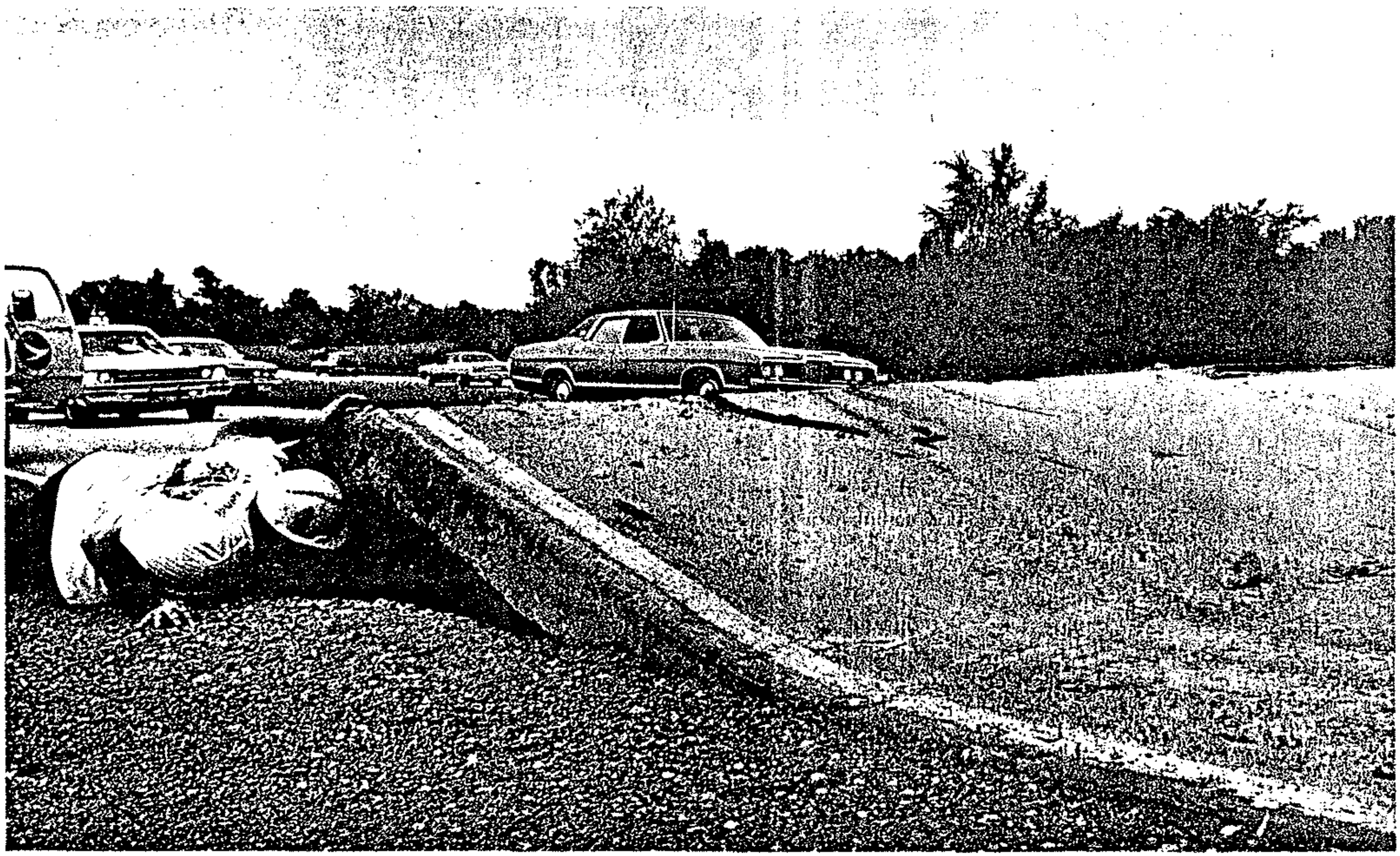


Figure 1. Blowup on Ohio 21, Summit County, 1975.
(Photograph, Akron Beacon Journal)

3. Develop and provide a set of guidelines for the design and installation of joint rehabilitation methods and techniques for utilization in a Pavement Management System.

Definitions

The following definitions are adopted for the execution of this project:

> Pressure Relief Joint- A transverse joint installed to relieve compressive stress for the purposes of reducing deterioration of existing joints, preventing blowups, and protecting abutments.
>
> Other Joint Rehabilitation Techniques- Any other technique to repair, replace, or restore an existing joint (such as precast or cast-in-place slabs, full-depth repairs, inverted "T" repairs, and prefabricated replacement joint assemblies) that may or may not be used along with pressure relief joints.

Scope

This study is confined to selected repair and prevention techniques for deteriorated transverse joints to increase the service life of existing concrete pavements. The useable end products from this study have been developed from analysis of extensive field performance data and the consensus recommendations of an advisory panel composed of experienced state highway pavement engineers possessing in-depth familiarity and expertise in concrete pavements.

The end products consist of extensive documentation of pavement rehabilitation performance and the development/updating of design guidelines and guide specifications for the following rehabilitation techniques:

1. Pressure relief joints.
2. Full-depth repairs.
3. Partial-depth repairs.
4. Joint resealing.

These results are ready for immediate implementation in existing pavements for further development and evaluation.

Research Approach

The objectives of the study were fulfilled through a combination of information gained from literature reviews, extensive field surveys and an expert panel of state highway engineers.

1. A literature review was conducted for pressure relief joints, full-depth repairs, partial-depth repairs and joint resealing. A bibliography is given at the end of this report.

2. A field survey of many rehabilitated projects was conducted and extensive condition, design, traffic and other data were

collected. Figure 2 shows the states where projects were located. These projects were evaluated by both the project staff and the expert state panel to extract as much information as possible from their performance. Many of the panel members were familiar with the individual projects, having been involved in their design and construction. Detailed comprehensive project evaluation reports were prepared for each of the 36 projects.

3. The expert state highway panel from eight states reviewed the evaluation of the individual projects and also provided extensive feedback on their recommendations for the design and construction of the four selected joint rehabilitation techniques.

This report was then prepared as a complete documentation of all of the above research work.

<u>Documentation Of Results</u>

The results obtained from this study are presented in the following sequence:

Pressure relief joints --
- Chapter II presents background information obtained from the literature review.
- Chapter III summarizes the evaluations of pressure relief joint projects.
- Chapter IV lists the conclusions and recommendations of the expert state panel.

Full-depth repairs --
- Chapter V presents background information obtained from the literature review.
- Chapter VI summarizes the evaluations of full-depth repair projects.
- Chapter VII lists the conclusions and recommendations of the expert state panel.

Partial-depth repairs --
- Chapter VIII presents background information obtained from the literature review.
- Chapter IX summarizes the evaluations of partial-depth repair projects.
- Chapter X lists the conclusions and recommendations of the expert state panel.

Resealing joints --
- Chapter XI presents background information from the literature review.
- Chapter XII summarizes the evaluations of joint resealing projects.
- Chapter XIII lists the conclusions and recommendations of the expert state panel.

Chapter XIV summarizes the results from the entire study.

Figure 2. States in Which Projects are Located.

The Design and Construction Guidelines and the Guide Specifications are placed in the Appendix so they can be extracted for use. A cost analysis for several selected projects is also documented in the Appendix.

Comprehensive project evaluation reports were prepared for each of the 36 projects included in this study. These individual reports are not published with this final report, but are available from the FHWA upon request.

CHAPTER II

PRESSURE RELIEF JOINT BACKGROUND INFORMATION

Pressure Damage

Description

Pressure damage includes spalling, crushing, or upheaval (blowup) of the concrete pavement at the transverse joints or cracks caused by expansion within the concrete pavement.

Pressure-induced spalls can be differentiated from other joint spalls by the size and shape of the spalled areas. Pressure-induced spalls are generally 6 to 12 in [152 mm to 305 mm] in length (measured from the joint or crack) and up to 12 in [305 m] in width (measured along the joint or crack). The depth of the spall typically varies from 1/2 to 2 in [13 mm to 51 mm] in depth (see Figure 3). Spalls caused solely by differential vertical movement across a joint or crack are often characterized by failures along a 45-degree plane and rarely extend more than 2 to 4 in [51 to 102 mm] from the joint or crack (see Figure 4).

Blowups are areas of localized upward movement (see Figure 5) or shattering of the pavement (see Figure 6) due to excessive compressive forces in the slab. They often occur in hot weather at a joint or crack which is closed or filled with incompressibles and will not allow expansion of the concrete slab.

Low-severity blowups produce minor buckling or shattering of the slab, but create little discomfort to the occupants of passing vehicles. Medium-severity blowups cause some discomfort, while high-severity blowups cause substantial discomfort, pose a safety hazard, and may require passing vehicles to reduce their speed. (29, 117)

Another problem caused by pavement growth is "bridge pushing." As a pavement expands during the warm season, and particularly where intrusion of incompressibles has occurred, the pavement will push against the bridge approach slabs. Incidents of cracked abutments and bridge decks being pushed nearly off of the abutments have been documented for both short- and long-jointed pavements. CRCP has been found to displace heavy anchor lugs due to high expansive stresses.

Many other types of secondary structures can also be damaged by pavement growth. These include manholes and other drainage and access structures in the pavement surface. They can be crushed, collapsed or rendered nonfunctional as they are moved by the pavement. Curbs and traffic islands are also subject to shattering, breakup, upheaval and failure as the surrounding pavement expands.

Pressure damage may continue after relief joints have been installed. The New York DOT has documented projects where the longitudinal joint ties have sheared as the traffic lanes moved independently (see Figure 7).

Figure 3. Photo of Pressure-Induced Spalls.

Figure 4. Photo of Spalls Caused by Differential Vertical Movement.

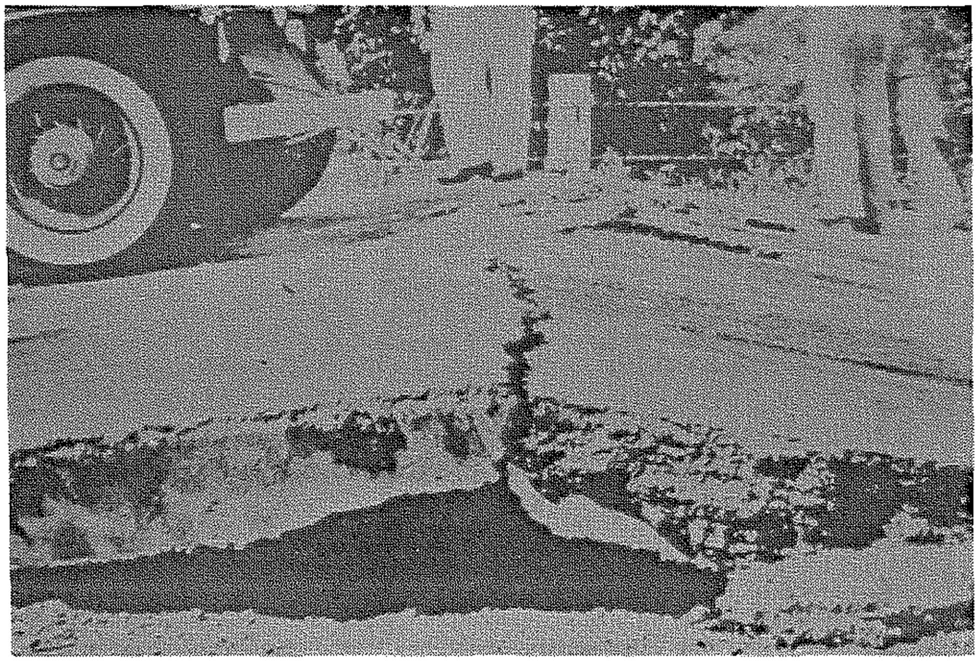


Figure 5. Photo of Blowup (Localized Vertical Movement).

Figure 6. Photo of Blowup (Shattered Pavement).

Figure 7. Photo of Traffic Lane Movement After Placement of Relief Joints.

Causes of Pressure Damage

The performance of concrete pavements in many areas of the country has been seriously impaired by expansive pressures caused by net increases in pavement length. This is caused by one or more of the following factors:

1. Surface intrusion of incompressibles into poorly sealed joints and cracks.
2. Pumping of base materials into joints and cracks from below.
3. Expansion of reactive aggregates in the PCC.
4. Extremely high pavement temperatures and moisture conditions relative to those that produce a neutral or "no stress" condition.

Incompressibles can infiltrate poorly sealed transverse joints and cracks when they are open. The joints and cracks open widest during the colder seasons, which are also the seasons during which sand and other deicing materials are placed on pavement surfaces. These materials enter the joints and cracks and prevent them from closing during warm seasons.

Intrusion can also occur from below the slab when vertical movements at the joints and cracks cause pumping. Water and base material particles are forced upward into the joints and cracks. In time, incompressibles can buildup (see Figure 8) which prevents the joint from functioning properly.

Pavement growth due to intrusion of incompressibles produces far more severe problems in pavements with long slab lengths. States such as California which have thousands of miles of short-jointed pavement (i.e., slab length less than 20 ft [6.1 m]) rarely experience blowups. States such as Illinois, Michigan and Virginia which have 40 to 100-ft [12.2 to 30.5-m] slabs frequently experience blowups (121).

Figure 9 illustrates the effect of slab length on joint movement for different temperature ranges. Longer joint spacings produce larger joint openings over a given drop in temperature. This in turn allows the intrusion of larger sizes and quantities of incompressibles from beneath the slab. The problem is magnified when the joint seals are not properly maintained or the seasonal thermal openings of the joints exceed the strain capacity of the joint sealing materials, which often happens in pavements with long slabs in harsh climates. Additional incompressibles may then enter from the surface. Moisture and thermal gradients warp and curl the pavement and result in additional joint movements.

Continuously reinforced concrete pavements (CRCP) do not normally exhibit blowups due to intrusion of incompressibles because their cracks are closely spaced and held tight by large amounts of reinforcing steel. These cracks generally do not fill with incompressibles and are not usually subject to significant spalling. However, blowups have occurred in CRCP when the reinforcing steel at a wide crack ruptures and the crack is infiltrated with incompressibles (72).

It should be noted that CRC pavements have pressure relief provided at terminal joints where the continuity of the pavement is interrupted for

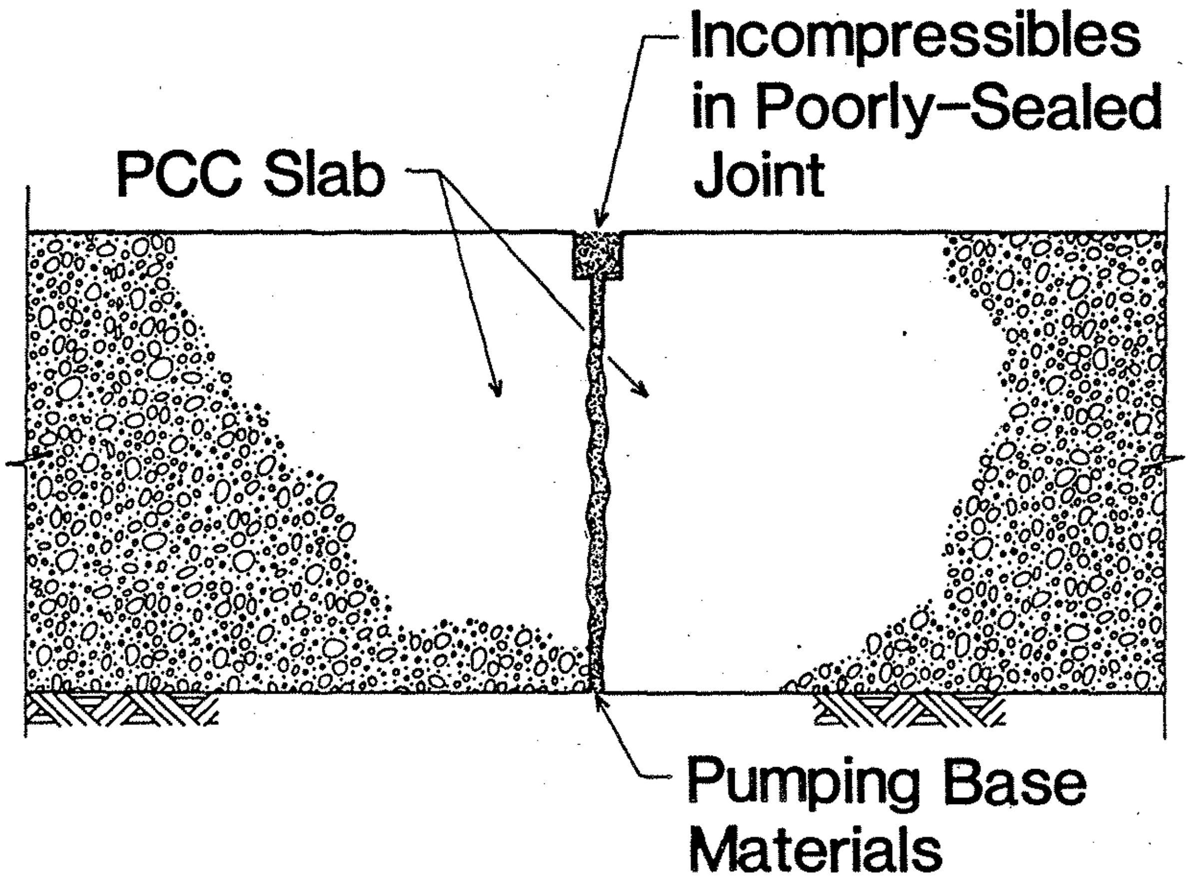


Figure 8. Intrusion of Incompressibles into Joint Area Through Poorly-Sealed Joint and Pumping of Base. (25)

Joint Opening - ins.		
	Stabilized Subbase	Granular Subbase
Joint Spacing-ft.	Temp Only	Temp Only
15	.040	.050
20	.050	.060
30	.080	.100
40	.100	.130
50	.130	.160
100	.260	.320

$\alpha = 5.5 \times 10^{-6}/°F$

$\Delta T = 60F$

Figure 9. Computed Maximum Joint Movement for Various Slab Lengths Over a Temperature Drop of 60°F [16°C].

bridges, drainage structures, junctions with other pavement types, etc. There have been cases where the wide-flange terminal joint design has failed, presumably due to compression from the concrete (35).

All three types of concrete pavements may experience blowups and other pressure damage due to the inclusion of expansive aggregates in the PCC mix and the presence of pavement temperatures that significantly exceed the ambient temperature that existed at the time of slab placement and curing.

The inclusion of reactive aggregates in the PCC mix can produce an actual increase in slab length. The increase in length is the result of a chemical reaction between alkalies in the cement paste and certain siliceous components of aggregates prevalent in many Western and Southwestern states. The product of the reaction is an alkali-silica gel which absorbs water and swells, causing cracking in the cement matrix. If sufficient free moisture is available, this gel can flow into the cracks and continue to swell, causing progressive expansion and cracking of the concrete. Visible signs of the alkali-silica reaction include map cracking and buildup of gel around aggregates and in cracks. If the expansion is unrestrained, it can cause a significant volume increase in the concrete, closing transverse joints and pushing bridge structures. If the expansion is restrained, longitudinal cracking, spalling or shattering of the concrete at the joints can occur. Pressure damage due to the alkali-silica reaction is a serious source of distress and structural failure in pavements and other concrete structures.

Another source of pavement "growth" is expansion due to increases in temperature above the neutral temperature, which is defined as the temperature at which the axial force in a pavement is equal to zero. For new pavements the neutral temperature is near the temperature at which the concrete solidified. Changes in moisture can be expressed as equivalent temperature changes. Kerr and Shade (64) have theorized that pressure damage occurs at a predictable increase in temperature above the neutral temperature. Thus, the placement of concrete pavements at higher temperatures may reduce the likelihood of blowups.

The result of concrete pavement growth is an increase in compressive stress in the slabs. When this stress exceeds the compressive strength of the slab at a given point, spalling or shattering of the slab occurs (63). Other factors that may cause spalling at the joint face include PCC fatigue (due to repeated loads applied through the dowels or repeated compressive stresses caused by thermal expansion), and corroded or improperly installed load transfer devices (especially dowels).

Spalling near the joints reduces the stiffness of the joints and introduces axial force eccentricities into the slabs, making them more susceptible to buckling or "lift-off" blowups (65). Spalling also increases the likelihood of shattered slab blowups because compressive forces must be resisted by smaller areas of concrete as spalling increases.

The correlation between spalling and blowup occurrence has been verified by studies conducted in Virginia (132), which indicate that as many as half of the joint faces involved in blowups exhibited prior deterioration. The results of a recent Michigan study (114) (see Figure 10) suggest that if a transverse joint is divided into five equal-length

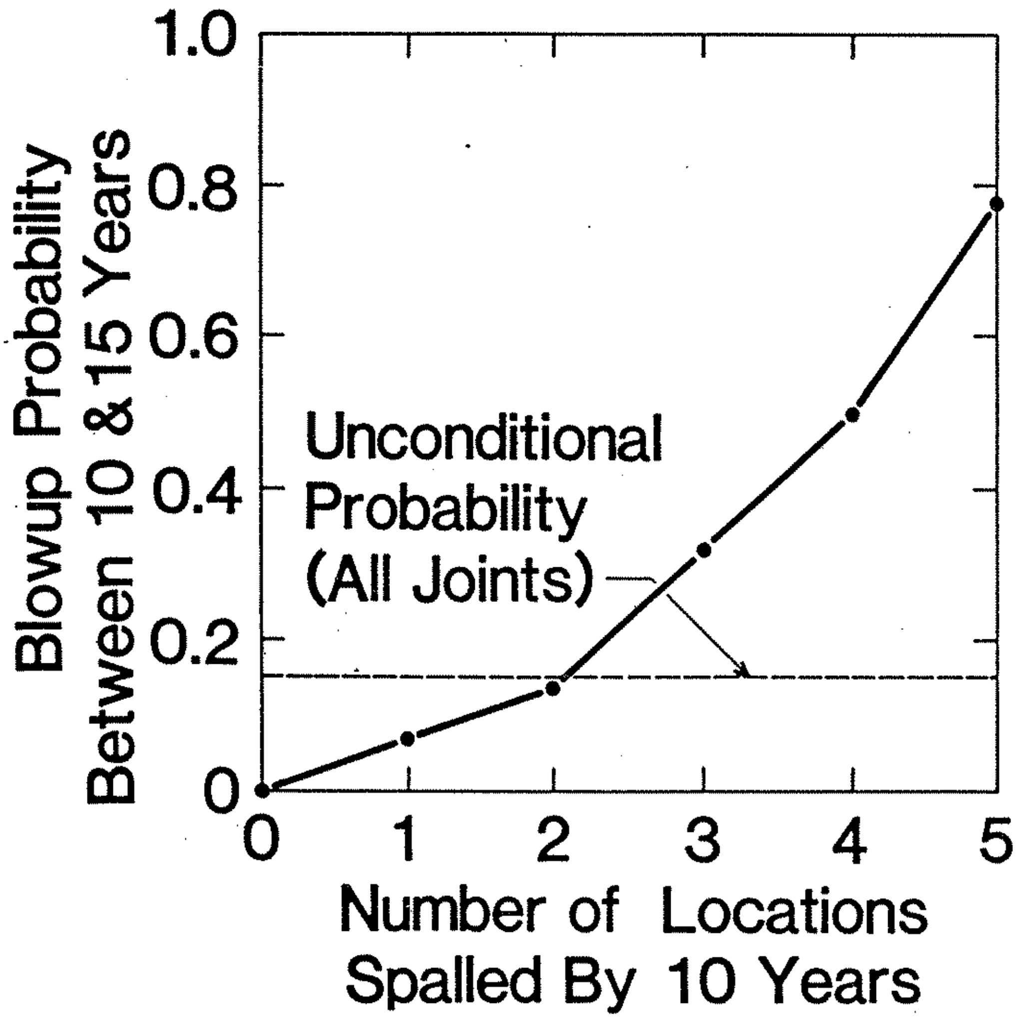


Figure 10. Probability of a Blowup as a Function of Joint Spalling for 100-ft [30-m] Joint Spacing (JRCP).

sections, the probability of a blowup occurring in the future increases greatly with the number of joint sections exhibiting spalling.

Several states have documented an increased occurrence of pressure damage in PCC pavements after bituminous overlays have been placed. It is believed that increased moisture, temperature and/or PCC deterioration may be responsible (43, 52, 53, 64, 106, 121, 142).

It is clear that many factors have a significant effect on the development of compressive forces and the capacity of the pavement to resist them. These factors include considerations in design, construction and environment and include the following (64):

1. Ambient temperature range at the project site.

2. Joint design and spacing.

3. Joint load transfer design.

4. Subdrainage design.

5. Steel reinforcement design.

6. Shoulder type (contribution to incompressibles).

7. Size and type of coarse aggregate used in PCC mix.

8. Concrete compressive strength (particularly near the joints).

9. Temperature of concrete at placement and during curing.

10. Slab moisture content.

11. Shearing resistance at the slab/subbase or subgrade interface.

13. Pavement age.

13. Type and condition of joint sealant present.

14. Use of deicers and deicing grits.

<u>Mechanisms of Blowups</u>

One theory of blowup development was proposed by Giffin (48) in 1943. According to this theory, the first stage of failure occurs prior to the actual blowup when compressive forces become severe enough to fracture the concrete below the surface (see Figure 11). The disintegrating concrete forms an inclined plane below the undamaged concrete. As the compressive forces increase, one slab moves up the inclined plane with sufficient force to shear the edge of the adjacent slab (Figure 12). This explains the observed blowup characteristic of one slab overriding the other.

Deterioration of the lower portion of the slab (due to "D" cracking, for example) also shifts the point of application of the compressive forces from middepth to some higher point. This results in the development of an effective moment at the joint (see Figure 13) which causes a "classic" or "lift-off" blowup.

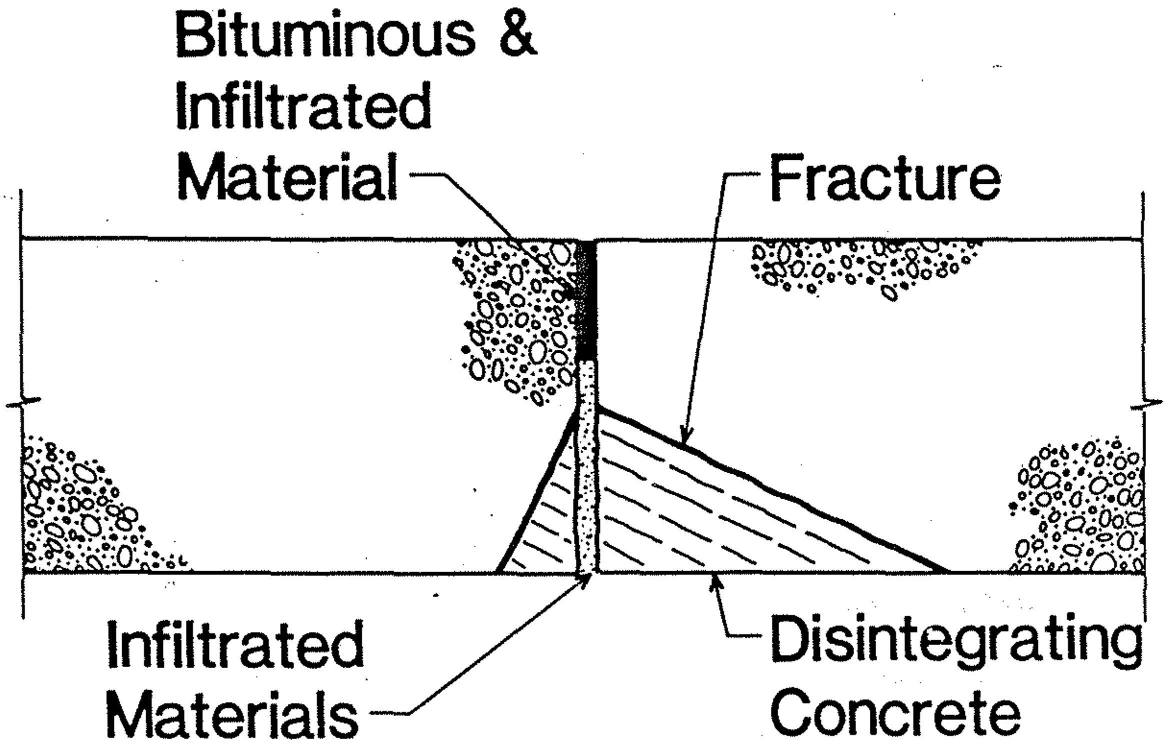


Figure 11. Fracture of Lower Portion of Slab Due to Infiltrated Materials (48).

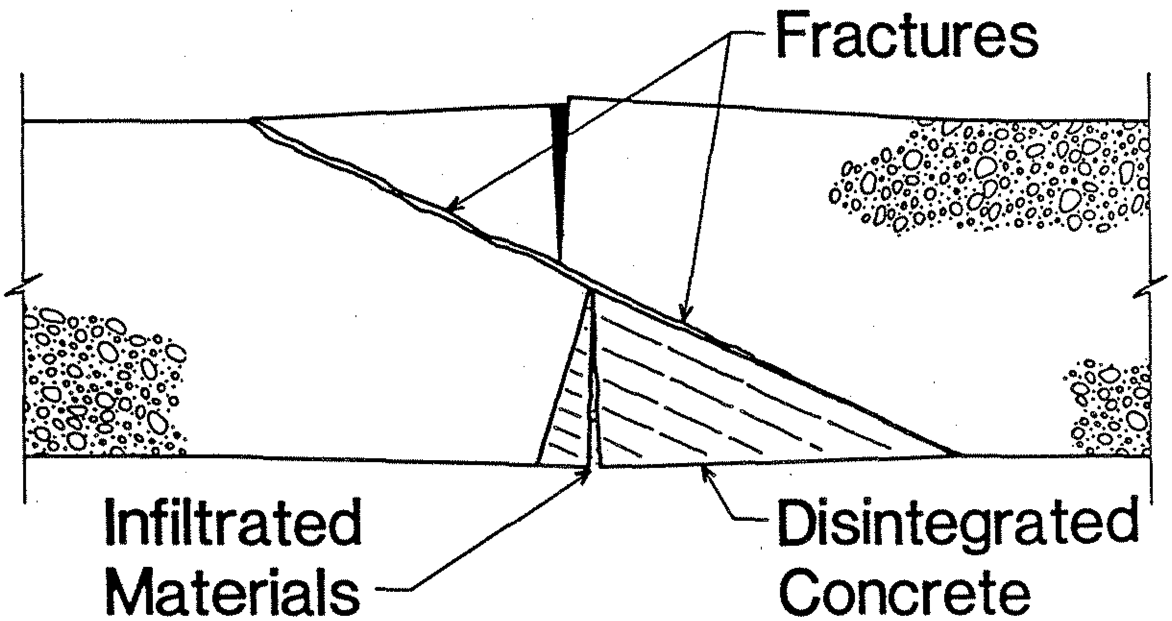


Figure 12. Development of Blowup Along Inclined Plane of Fracture (48).

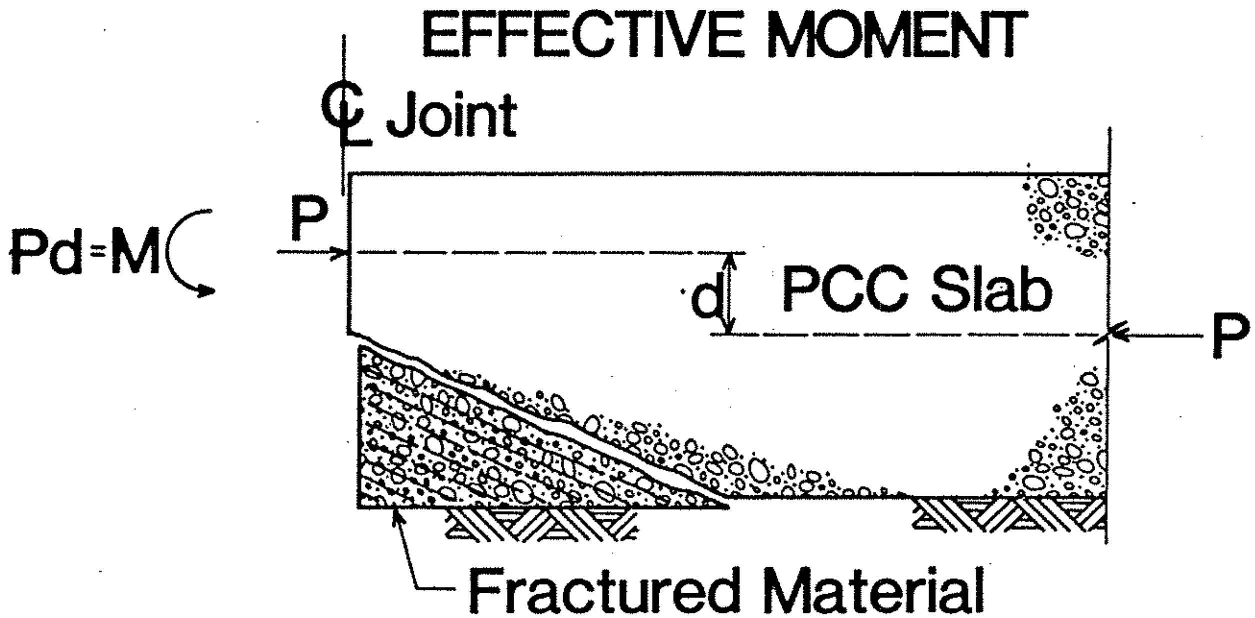


Figure 13. Development of an "Effective Moment," Resulting in a Classic Blowup (48).

Kerr and Shade have suggested the mechanism shown in Figures 14 and 15 for "classic" or "lift-off" blowups (64). The pavement is treated as a concrete beam with a hinge; movement of the hinge is restricted to the negative "z" axis. Temperature and moisture increases induce an increase in axial compression forces (N_t) in the pavement slab. If the stiffness of the joint is reduced (through spalling) and/or the compression is sufficiently large, the beam buckles vertically, thereby reducing axial compression in the area of buckling. In the areas adjacent to the blowup, the compression varies according to factors such as slab/subbase friction, but is less than the critical value that caused buckling.

Analyses were developed to predict allowable safe temperature increases over a pavement's neutral temperature. These analyses consider the pavement thickness, sliding frictional resistance between the slab and subbase, effective flexural stiffness of the pavement, coefficient of linear thermal expansion, and rotational and axial stiffness of the joints and cracks (63, 64, and 65).

Expansion Joints

Expansion joints are constructed to relieve compressive stresses in the slab caused by thermal- or moisture-related growth, reactive aggregates, or intrusion of incompressibles into joints. Expansion joints are constructed by cutting or forming a gap (which is filled with a compressible material) through the depth of the slab to permit expansion to take place. These joints have no aggregate interlock to provide load transfer, and thus tend to be susceptible to pumping and high deflections.

Such joints in new pavement are very difficult and expensive to construct and often show poor performance. For these reasons, nearly all agencies have limited the use of expansion joints to placement in existing pavements which have exhibited pressure damage.

Expansion Joints in New Construction

Expansion joints constructed in new pavements usually employ a dowel system to transfer load and compressive joint filler to permit expansion (see Figure 16). Their width is often 1 in [25 mm], although this will vary from agency to agency. Commonly used filler materials include bituminous materials, closed cell foam, styrofoam, and cork.

Dowels used for load transfer are generally smooth and are fitted with an expansion cap to provide room for the dowel bar to move during expansion (143). The dowels must be painted, encased in plastic or epoxy or formed from noncorrosive materials, and lubricated to ensure horizontal movement.

The seals used in these joints should be well-maintained to prevent intrusion of incompressibles and infiltration of water. The sealant material must be selected for extensibility to accommodate large horizontal joint movements and for strength to resist intrusion of incompressibles.

Many states no longer use expansion joints in new construction due to high cost, poor load transfer and resulting distresses (e.g., pumping, faulting, spalling, etc). However, New Jersey continues to construct expansion joints rather than contraction joints, and Michigan installs expansion joints at every eighth joint (41-ft slabs [12.5 m]) if the project is constructed between September 15 and April 15.

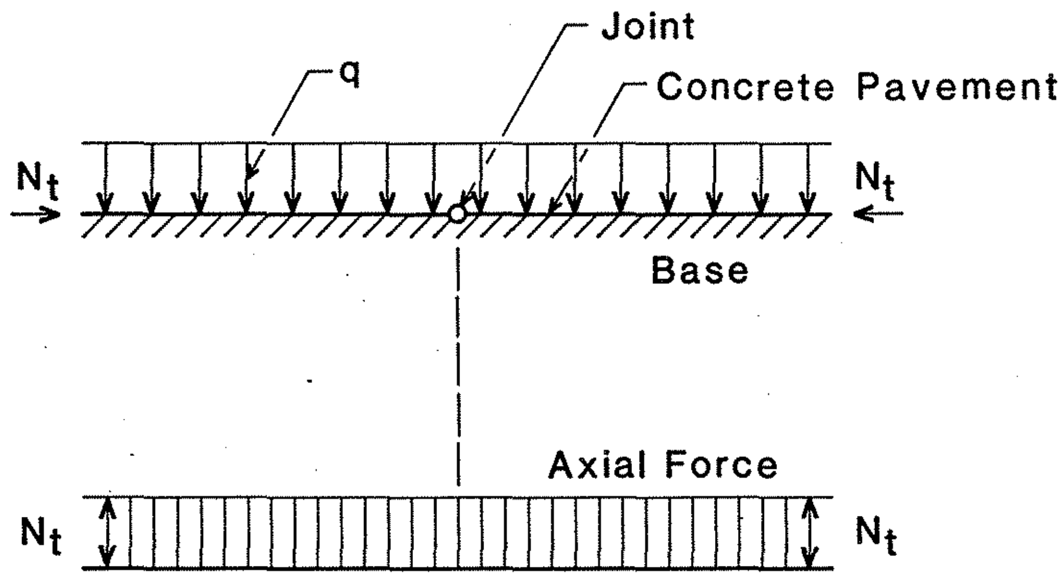


Figure 14. Mechanism for "Classic" or Lift-off" Blowups-forces (64).

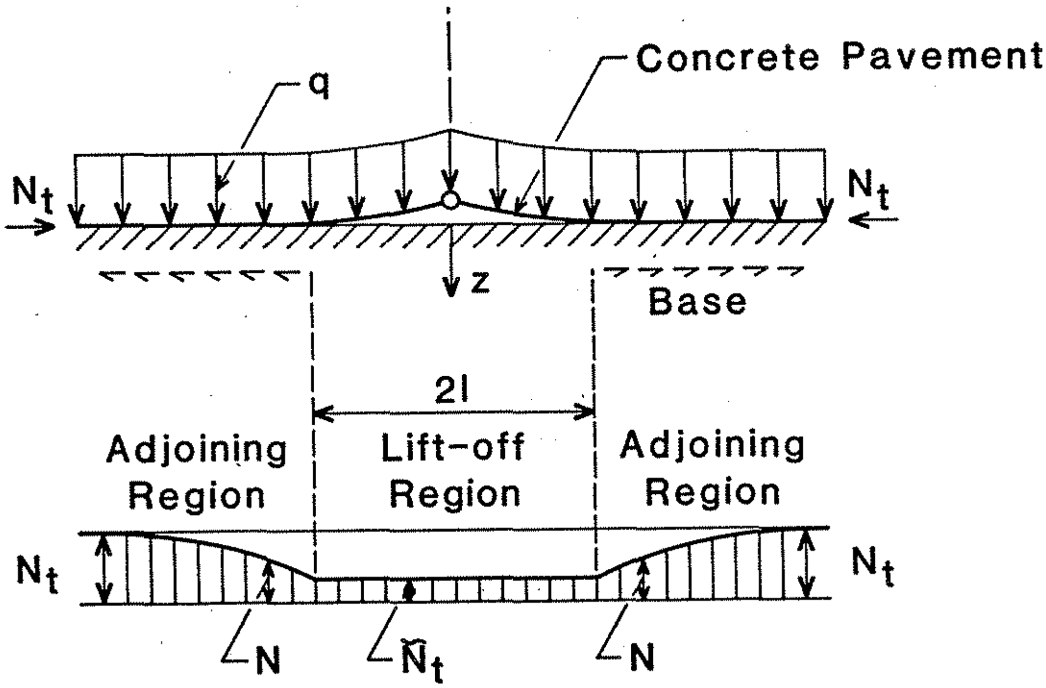


Figure 15. Mechanism for "Classic" or "Lift-off" Blowups - Movement (64).

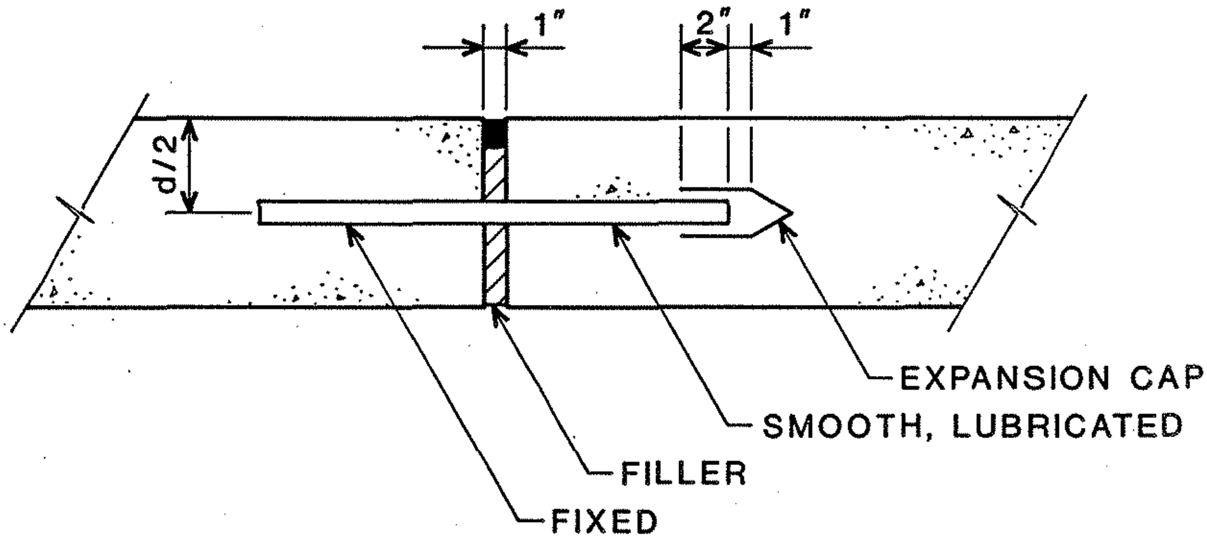


Figure 16. Illustration of Expansion Joint in New Pavement [1 in = 25.4 mm].

Pressure Relief Joints

Description

The function of pressure relief joints is to relieve compressive stress in order to prevent blowups, shattered slabs, severe joint spalling and damage to secondary structures. Although designs vary, they are normally 2 to 4 in [51 mm to 102 mm] wide.

Pressure relief joints are usually filled with a compressible filler material such as styrofoam or sponge rubber to prevent intrusion of incompressibles, although asphalt concrete filler has been used by some agencies. Preformed joint seals have been used by some agencies with limited success.

Appropriate Uses and Locations

Net increases in concrete pavement length are often caused by intrusion of incompressibles into poorly sealed joints, pumping of base materials into joints, and/or expansion of reactive aggregates in the PCC. If untreated, these length increases will create compressive forces in the concrete pavement and cause blowups and/or bridge pushing. Pressure relief joints reduce these pressures and, when used appropriately, can prevent pressure damage for several years.

Because they provide no load transfer, deflections at the relief joint tend to be high. Significant pumping, faulting, corner breaks, and slab deterioration can thus occur in the vicinity of a pressure relief joint. Pressure relief joints should be used only on pavements which have experienced blowups or are pushing bridges.

The optimum time for construction of pressure relief joints is not known. Many jointed concrete pavements perform for over thirty years without experiencing blowups or bridge abutment pushing. Pressure relief joints can cause loss of load transfer in adjacent joints, further intrusion of incompressibles, widening of joints/cracks, slab faulting, and overall accelerated pavement deterioration. Construction of expansion joints is currently recommended <u>only</u> when major blowups or other pressure-related damage has occurred.

Studies by several agencies have concluded that blowups tend to relieve stress for about 500 ft [152 m] on either side. For this reason, pressure relief joints are typically installed at intervals of 700 to 1500 ft [213 to 457 m]. When bridge pushing is the only problem, they are typically located <u>only</u> near the approach slabs.

Pressure relief joints are often ineffective when placed near other pressure-relieving devices. Bridge approach expansion joints will usually provide sufficient pressure relief within 500 ft [152 m] and additional relief is not needed. Pavements which have sustained full-width blowups may not need pressure relief joints within 500 ft [152 m] of the blowups, especially if the blowups were patched with bituminous materials. Wider-than-normal joint openings in the area of a blowup indicates that further relief is not needed.

Because of the difficulty in sawing through dowels or other load transfer devices and the danger of encountering unstable subbase conditions near old joints, pressure relief joints are often placed near mid-slab. Some agencies have placed them as one joint of a full-depth repair; however, this often produces repair rocking and accelerated failure unless mechanical load transfer devices are provided.

Design, Materials and Construction

Nonbituminous-Filled Relief Joints Without Load Transfer

Although exact construction procedures vary among agencies, two methods are generally used. Diamond blade saws are used to make two full-depth cuts about 4 in [152 mm] apart. The material between the cuts is removed using light jackhammers if necessary. The faces of the joint are cleaned and the filler material is installed.

Carbide tooth wheel saws (sometimes called "rock saws") are also used to make one full-depth cut about 4 in [152 mm] wide across the pavement. The faces are cleaned as necessary and the filler material installed. The use of these saws should be considered carefully, since they may produce a large amount of spalling at the pavement surface.

The joint is usually filled with a compressible filler material such as styrofoam or sponge rubber to prevent the intrusion of incompressibles, although asphalt concrete filler has been used by some agencies. Preformed joint seals have also been used with some success. These compressible filler materials are often coated with a lubricant-adhesive which facilitates installation of the filler and holds it in place after installation. Special hydraulic equipment is often used to compress and install the filler. Figure 17 presents a cross-section of a typical nonbituminous-filled relief joint without load transfer.

Bituminous-Filled Relief Joints Without Load Transfer

Some agencies have constructed asphalt concrete patches in Portland cement concrete pavements to serve as expansion joints. These patches are 3 to 4 ft [0.9 to 1.2 m] long and are often placed in deteriorated areas which would otherwise require a full-depth PCC repair. Pavement removal and construction methods are similar to those used in normal bituminous patching operations.

Asphalt concrete has also been used as filler material in the four-inch [102 mm] pressure relief joints described previously; however, it is stiffer than most filler material and will not accommodate as much movement as other filler materials.

The use of asphalt concrete-filled relief joints and patches in expanding PCC pavements has often resulted in "humping" of the asphalt patch as the concrete pavement expands. "Humping," loss of load transfer, slab rocking, and settlement or heaving of asphalt patch areas have resulted in rough pavements and loss of pavement serviceability. When patches of this type have been placed in only one lane, blowups have often occurred in adjacent lanes.

Bituminous-filled Relief Joints With Load Transfer

Figure 18 illustrates a retrofit relief joint design used by the Ohio Department of Transportation. A 1- to 3-ft [0.3- to 0.9 m] section of

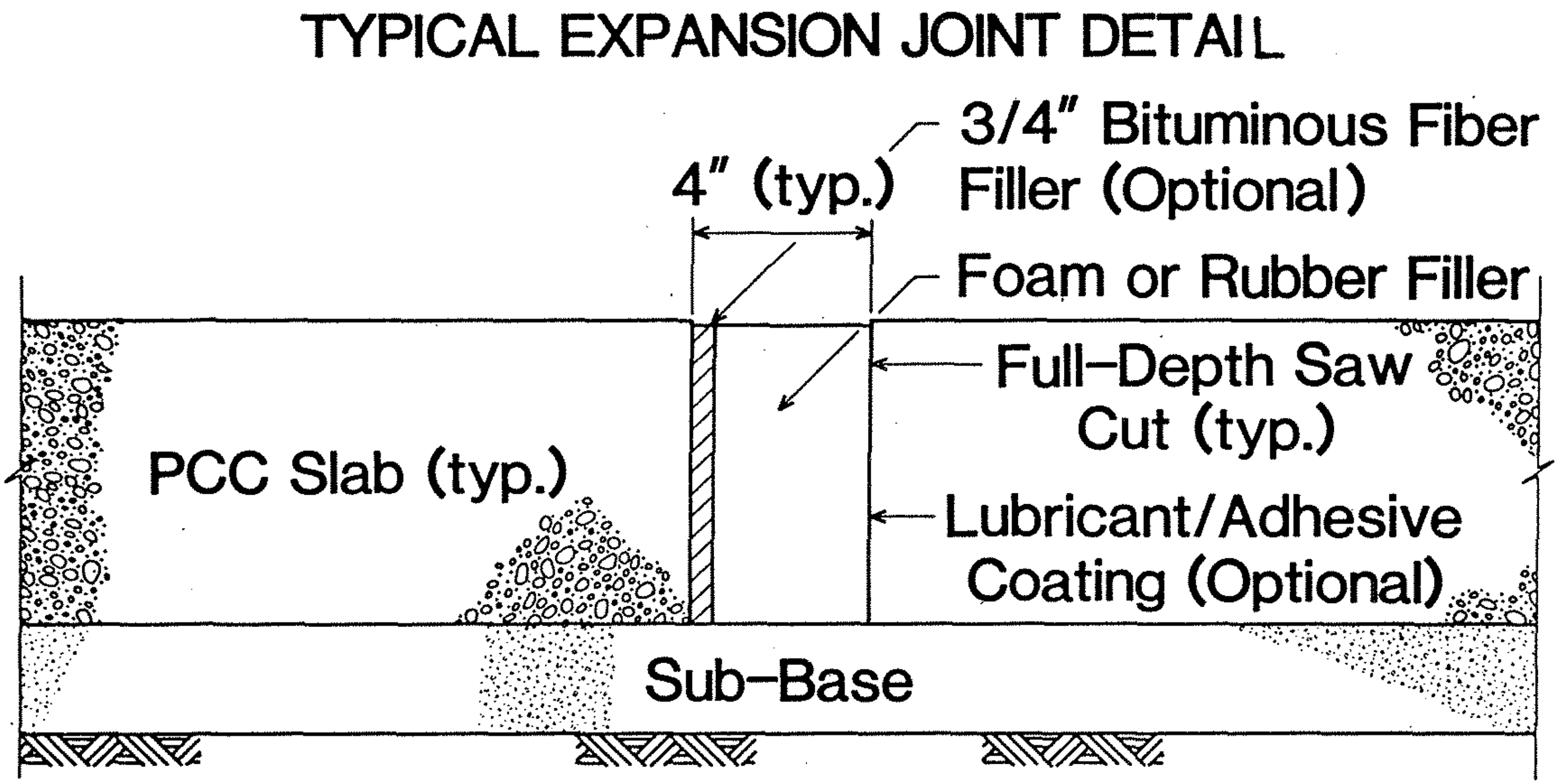


Figure 17. Cross-section of a Typical Non-Bituminous - Filled Pressure Relief Joint Without Load Transfer [1 in = 25.4 mm].

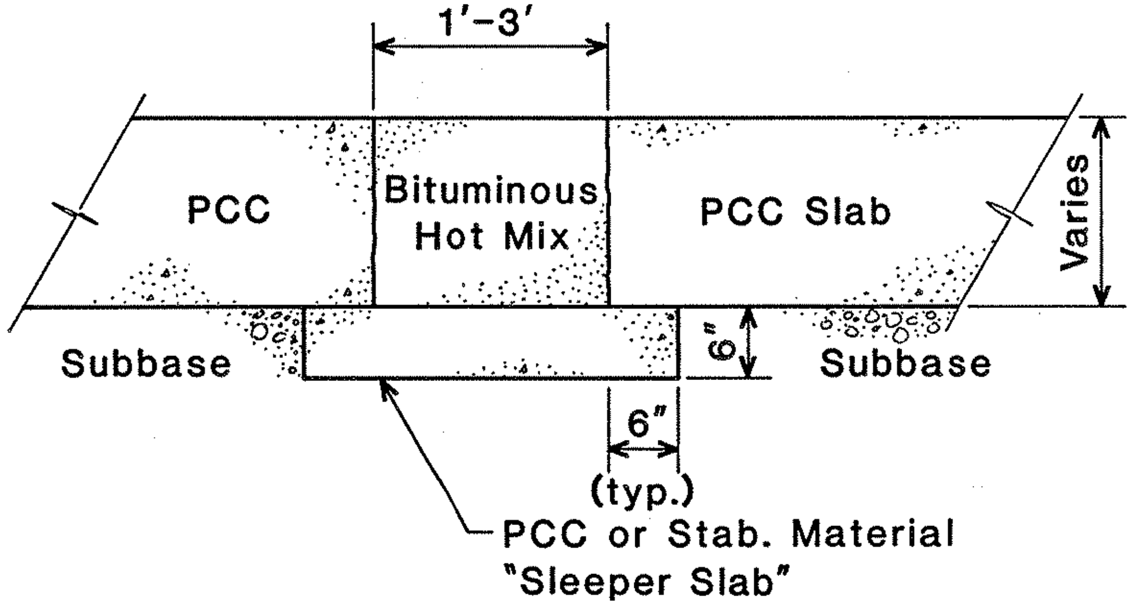


Figure 18. Ohio DOT Pressure Relief Joint Design
[1 in = 25.4 mm].

Portland cement concrete pavement is removed and 6 in [152 mm] of subbase is excavated between the sawcuts and under the existing slab a distance of 6 in [152 mm]. The excavated subbase is replaced with PCC concrete or a stabilized base material prior to filling the patch with asphalt concrete. The resulting load transfer system is similar to that provided by an "inverted tee" patch, and the problems associated with its construction and performance are similar as well (i.e., poor consolidation of concrete under the existing slab, differential heaving or settlement of the patch, and one-directional load transfer). "Humping" of the bituminous material is still a potential problem with this approach.

Expansion Joints Constructed Within Full-Depth PCC Repairs

Special heavy-duty expansion joints with dowels for load transfer have been used on highways with heavy traffic, particularly where a bituminous overlay is to be placed. An example design used by the Illinois Department of Transportation is shown in Figure 19.

These expansion joints are usually placed in the center of a full-depth repair. The repair is tied to the original slab. The expansion joints are prefabricated assemblies which include dowels, chairs and a fibrous filler material. The dowels are capped and coated in a similar manner to those used in new construction. While this is an expensive design, it does provide load transfer and reduce the occurrence of localized distress in subsequent bituminous overlays placed over the expansion joint.

Some agencies have constructed expansion joints at one or both edges of full-depth PCC repairs. This design has often produced rocking, premature cracking and spalling of the repair because of poor load transfer. Some success in minimizing this damage has been achieved when appropriately sized dowels have been included in the joint design. Undercut repairs (with and without dowels) have also been constructed with relief joints to provide load transfer and to accommodate pavement growth (see Figure 20).

Past Performance of Pressure Relief Joints

The performance of pressure relief joints provides an indication of their effectiveness. Pressure relief joints installed in sections of pavement experiencing appreciable compressive stress will begin to close almost immediately. Figure 21 (97) illustrates experience with closure in Nebraska where reactive aggregate exists in the pavement.

The data show that most of the closure experienced during the first year occurred before the summer months, when unrelieved stresses would be highest. It was concluded that pavement stresses, even in the winter, are often too high to be relieved entirely by the natural thermal contraction of the pavement.

The movement of intermediate joints within a typical pavement section with pressure relief joints is shown in Figure 22. As expected, the movement was greatest near the pressure relief joints, gradually decreased toward the center and was practically zero at the center.

Analysis of this data was used to determine that the pressure relief joints would have been effective over a distance greater than 1000 ft [305 m], at least for this level of slab/subbase interface friction

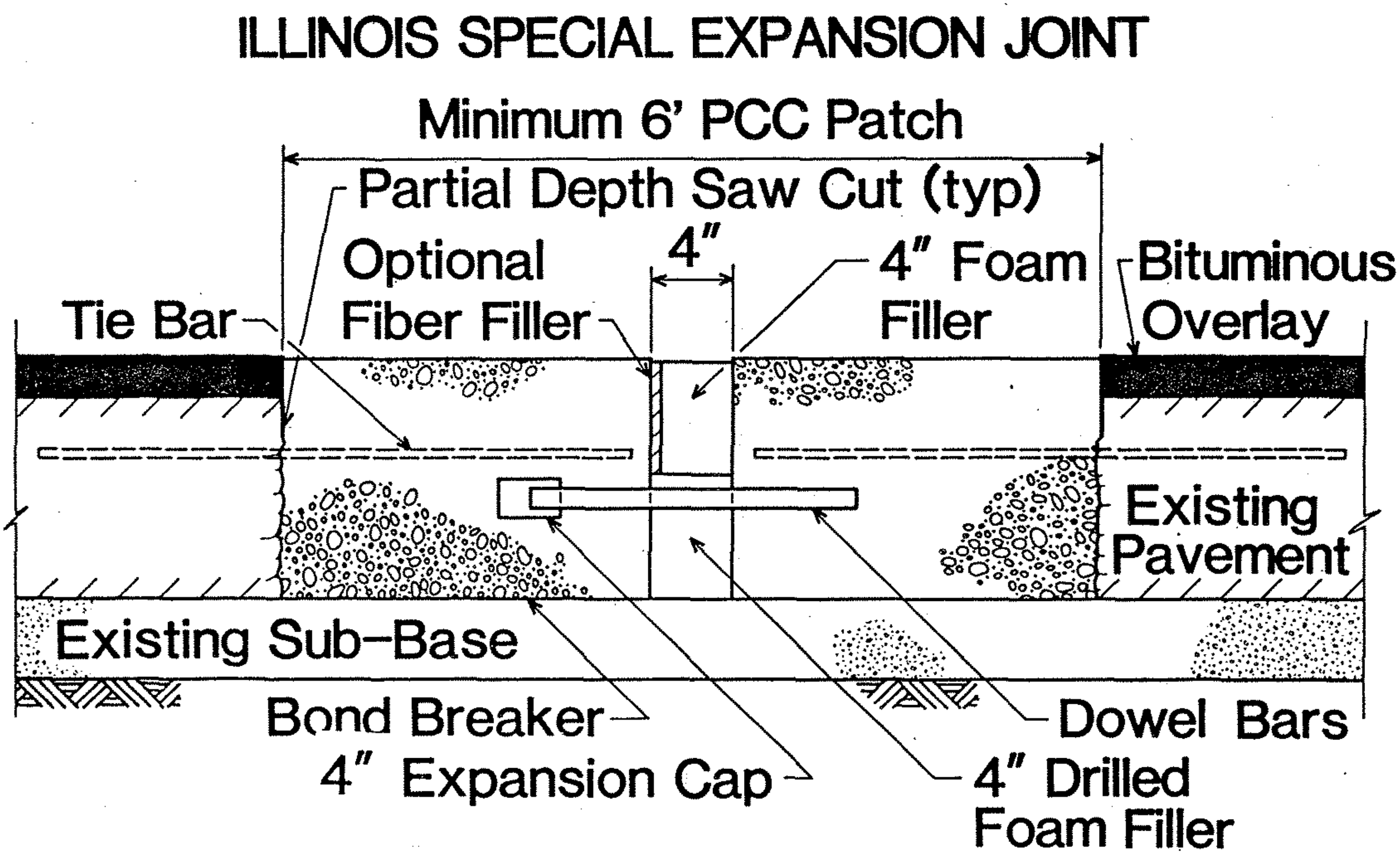


Figure 19. Illinois DOT Heavy-Duty Pressure Relief Joint Design [1 in = 25.4 mm].

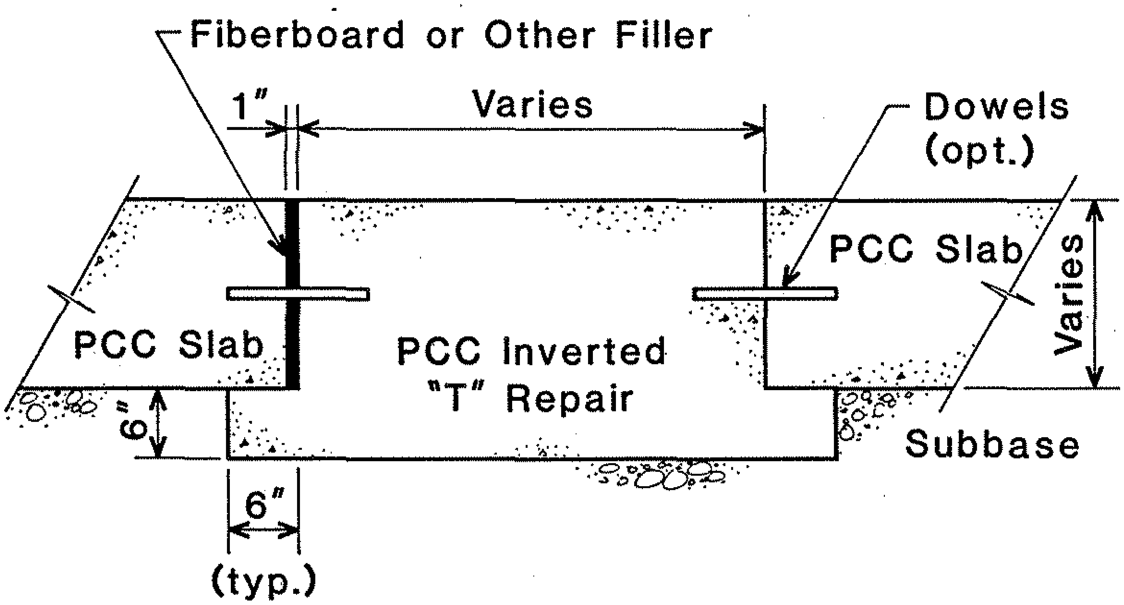


Figure 20. Illustration of Inverted-Tee with Pressure Relief
[1 in = 25.4 mm].

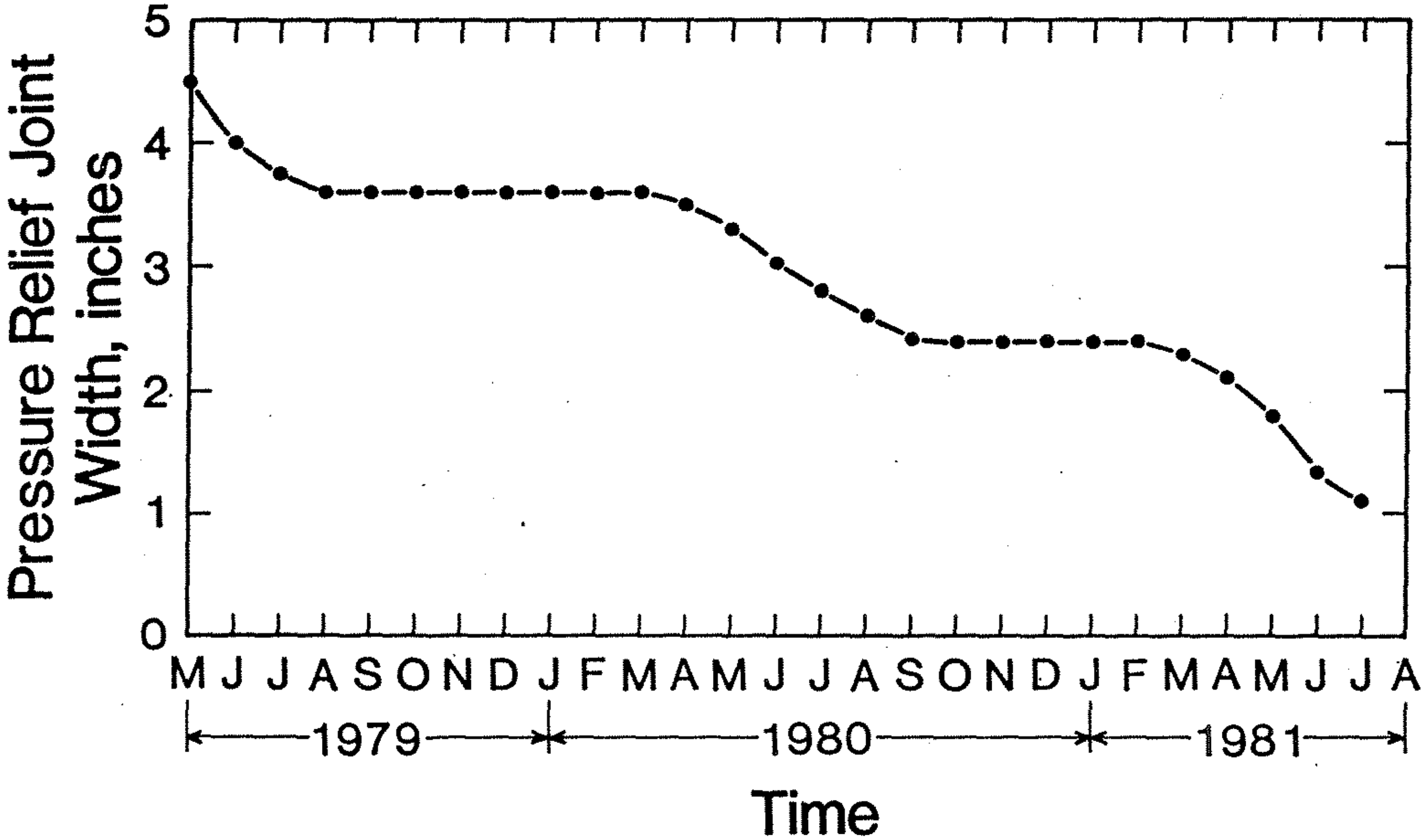


Figure 21. Illustration of Pressure Relief Joint Closure Over Time (97) [1 in = 25.4 mm].

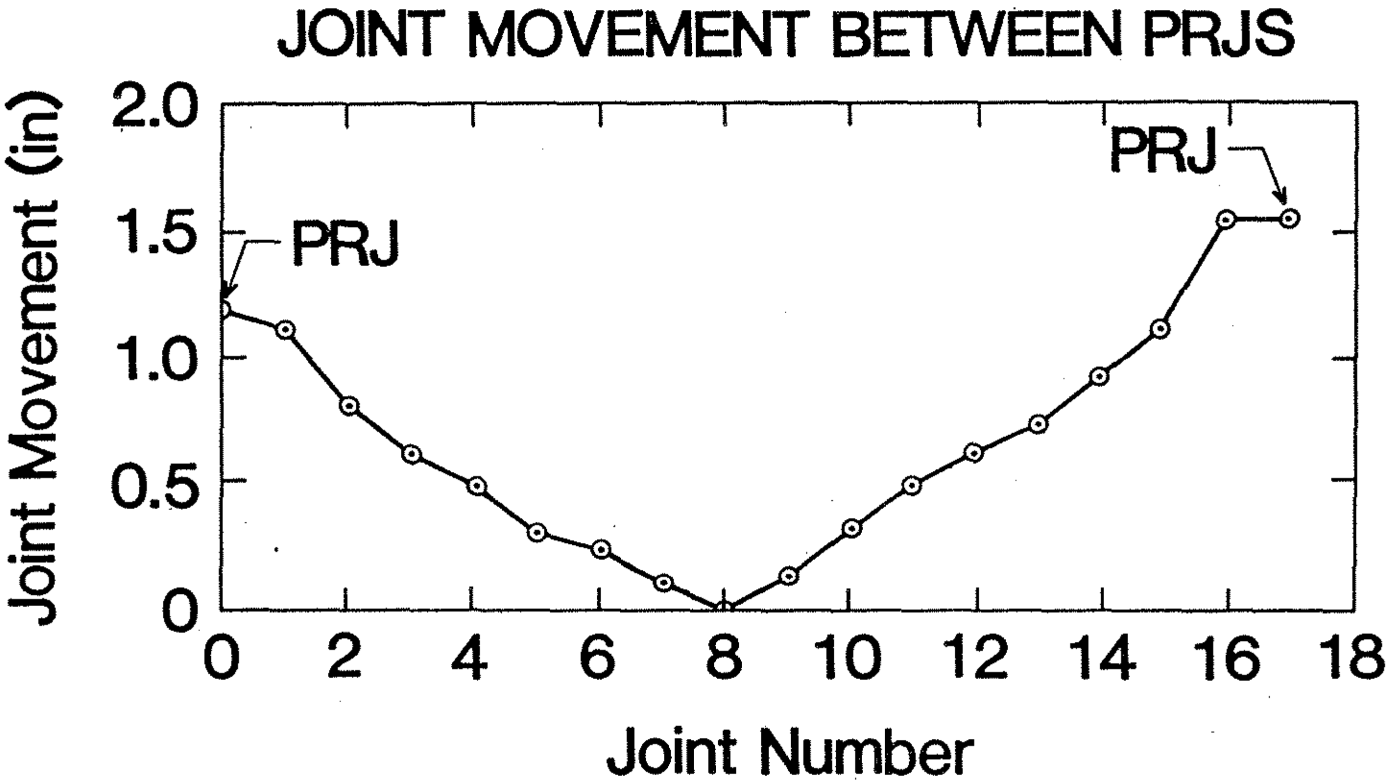


Figure 22. Illustration of Movement of Intermediate Joints [1 in = 25.4 mm].

(granular base). This is based on the assumption that pressure relief joints are effective until more than one joint away from the relief joint remains stationary. Determining the maximum effective length is difficult, due to the inverse relationship between effective length and compressive stress within the pavement. The more residual compressive stress a pavement contains, the greater the length of pavement over which the relief joint is effective; conversely, when the compressive stresses are small the distance over which the joint is effective is shorter.

Figure 23 presents the results of studies performed in Virginia to evaluate the effectiveness of 4-in [152 mm] wide pressure relief joints (installed at 1000-ft [305-mm] intervals in a jointed reinforced concrete pavement) in preventing blowups (75, 76). During the summer of 1974 (before the installation of pressure relief joints) the 15-mile [24-km] segment of pavement experienced 24 blowups. No blowups occurred during the summer of 1975 after installation of the relief joints. Whether this was due to the installation of the relief joints or the relief of all existing pressure in the pavement by the 24 blowups is subject to speculation.

A comparison of the occurrence of blowups prior to and after construction of pressure relief joints was used to develop a relationship between blowups and pavement characteristics. Analysis of pavement distress found in the summers of 1974 and 1975 also indicated that relief of compressive stress (by either the blowups or the pressure relief joints) reduced the occurrence of other distresses such as spalling and cracking. Whether or not the additional pressure relief was needed is difficult to determine. Significant closure of the relief joints did occur, but significant opening of other joints near the relief joints also occurred.

A major problem of pressure relief joints is load transfer. Ohio has conducted deflection testing on numerous in-service pressure relief joints and determined an average load transfer of 51% for a 4-in [102-mm] wide, undowelled relief joint and an average load transfer of 74% for a 1-in [25-mm] wide, dowelled relief joint. The amount of this load transfer will influence the amount of faulting that will occur at the joint. Figure 24 compares the faulting of a dowelled contraction joint (obtained using a regression equation from Reference 29 for a typical Illinois pavement) and the faulting of several in-service, undowelled pressure relief joints as a function of 18-kip [80-kN] equivalent single axle loadings (ESALs).

Installation of pressure relief joints has been accompanied by accelerated pavement deterioration on some projects. For example, when pressure relief joints were installed in only one direction of a divided, jointed reinforced concrete pavement section in Michigan, a condition survey conducted five years later revealed that the side with pressure relief joints had deteriorated to a greater extent. This demonstrates a major problem to be solved in the design of pressure relief joints--they must stop blowups without causing other types of distress.

Limitations of Pressure Relief Joints

As pressure relief joints have been constructed, it has become apparent that certain precautions are necessary to achieve desired results. Some of these precautions and their related problems are discussed below.

Project	Number of Blowups		
	To Feb. '74	w/o Relief Summer '74	With Relief Summer '75
1-NB	25	8	0
1-SB	29	5	0
2-NB	18	0	0
2-SB	18	4	0
3-NB	3	5	0
3-SB	2	2	0
	95	24	0

Figure 23. Results of Virginia Study.

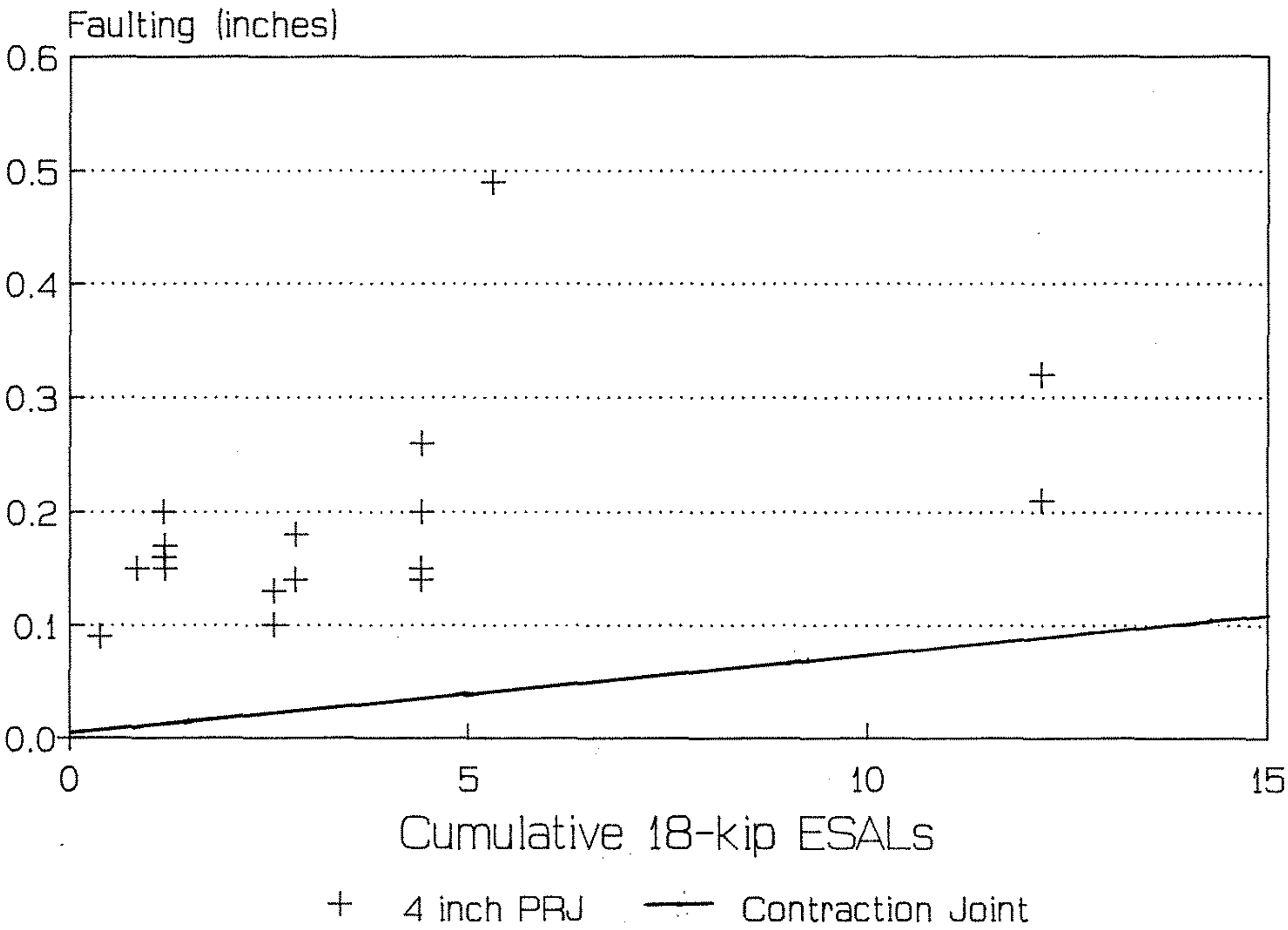


Figure 24. Faulting of Dowelled Contraction Joint and Undowelled Pressure Relief Joint [1 in = 25.4 mm].

General Limitations

Because they are usually constructed with no load transfer, pressure relief joints should be used only on pavements which have experienced blowups or are pushing bridges. Deflections at pressure relief joints will tend to be high, the adjacent slab may deteriorate, and the joint may pump and fault. It is recommended that if pressure relief joints are placed adjacent to bridges where pushing is a problem, load transfer should be established on plain jointed undowelled pavements for 6-10 contraction joints from the pressure relief joint.

Pressure relief joints allow the pavement to move toward the joint (see Figure 22). All aggregate interlock load transfer may be lost at joints within the effective range of the expansion joint. For this reason, pressure relief joints are not recommended for short-jointed concrete pavements without load transfer devices except adjacent to bridges where pushing is a problem.

Effects of Pressure Relief on Existing Pavement Design

Short-jointed undowelled concrete pavements are poor candidates for the use of pressure relief joints. Pressure relief joints will allow the joints to open, resulting in loss of aggregate interlock load transfer in the area of relief, which may also cause slab cracking and faulting. They will also allow water to enter the pavement structure, resulting in deterioration of the subbase, pumping, and rocking of the slab. Similarly, pressure relief joints should not be used in continuously reinforced concrete pavements because they destroy the continuity of the pavement and allow water to enter the subbase, resulting in rapid loss of subgrade support. They should be used on these types of pavements only near bridges when shoving is a problem or if blowups have occurred.

The effect of pressure relief on existing joint seals must also be considered. The installation of pressure relief joints produces an undesirable side effect of opening intermediate joints so widely that preformed compression seals lose contact with joint reservoir walls. This creates a process in which particles and water are free to infiltrate the joint, inhibiting joint closure and creating additional compressive stress in the pavement. Similarly, the effectiveness of other types of joint seals will be diminished if they are damaged by excessive joint openings.

It must be confirmed that transverse joint seals will remain effective after installation of pressure relief joints or it may be necessary to reseal the joints. Since it is impossible to predict where excessive openings will occur, it is recommended that test relief joints be installed prior to full pressure relief in pavements with compression seals.

Use on Multi-lane Pavements

Pressure relief joints are normally installed on pavements with more than one traffic lane and it is frequently impossible to install the joint across the full pavement width in one day. When relief is provided for one lane only, the other lane(s) can be subjected to higher compressive stresses and blowups or shearing of the longitudinal tiebars can result (see Figure 25). Thus, it is necessary to install pressure relief joints in all adjoining lanes as soon as possible. If the joints are placed during seasons with small daily temperature variations, a maximum period of 48 hours between installation of expansion joints in adjacent lanes should not be harmful.

Figure 25. Blowup in Inner Lane Due to Relief Joint Placed in Outer Lane (Courtesy of Kentucky Department of Highways).

In cases where the unrelieved lane(s) is of good-quality concrete, restraint between the lanes has been observed to prevent functioning of the relief joint so that the joint filler material is not held tightly in position and can float out during a heavy rain. Such an event can be avoided by installing the joint full width within 48 hours.

Installation in Hot Weather

During the warmest time of the year, or in pavements with reactive aggregates, compressive forces in the pavement may be sufficient to pinch or bind the saw blades during the sawing operation. In addition the problem of unequal pressure between adjacent lanes is often aggravated during warm weather. For this reason, installation within a temperature range of 40 to 70°F [4 to 21°C] is recommended. Some agencies have been able to install relief joints during the summer months by sawing at night or early in the morning.

Complete Relief Joint Closure

Pressure relief joints may close completely over time, making the pavement susceptible to blowups and bridge pushing again. If intrusion of incompressibles into the joints is not stopped or if reactive aggregate expansion is progressing, the construction of pressure relief joints will provide only a temporary solution. When the joints have closed, new pressure relief joints will be needed.

Pressure Relief When Overlays Are To Be Placed

On some pavements, blowup frequency has increased after overlay with bituminous materials, indicating a need for pressure relief prior to overlay. However, when expansion joints are placed prior to overlay, the overlay often deteriorates badly under heavy traffic in the area of the expansion joint. This is due to vertical shearing caused by high differential deflections at the joint. It has also been determined that the placement of bituminous overlays aggravates problems already inherent in some blowup-susceptible pavements by holding moisture in the concrete pavement structure (e.g., accelerated freeze-thaw deterioration of porous aggregates resulting in weakened concrete near the joint areas) (52, 53).

Installation of pressure relief joints prior to placement of bonded concrete overlays has produced debonding in the area of the joint. This is because it is extremely difficult to saw the joint in the overlay soon enough to prevent the underlying slab from moving independently of the overlay. It is recommended that if pressure relief is needed, construction of pressure relief joints be done <u>after</u> construction of the bonded concrete overlay.

CHAPTER III

EVALUATION OF IN-SERVICE PRESSURE RELIEF JOINT INSTALLATIONS

This chapter presents summaries of the design and performance of several projects where pressure relief joints have been installed. Detailed evaluation reports have been prepared for each of these projects and are available from the FHWA upon request. The first two letters of the identification indicate the state (e.g., IL), the next three numbers indicate the highway number (e.g., 072 is I-72), and the last three numbers provide the milepost by which the project is identified.

Overlaid Relief Joints (Wide--Several Feet)

IN069064 - The original pavement was 10-in [254 mm] JRCP with 40-ft [12.2 m] contraction joints and was constructed and opened to traffic in 1964. Three-foot [0.9-m] wide, asphalt concrete-filled pressure relief joints were installed in 1975 in five experimental sections using three different relief joint spacings: 1000 ft [305 m], 500 ft [152 m] and 40 ft [12 m]. One 1/2 mile [805 m] section served as a control and contained no relief joints. The pressure relief joints were installed because of bridge pushing, in anticipation of pressure buildup problems, and as part of IDOH policy to prevent blowups of overlaid "D" cracked pavements. Partial-depth repairs were placed at selected joints at the same time to address spalling due to "D" cracking. A 4.25-in [108 mm] asphalt concrete overlay was placed immediately afterward.

By 1985, the overlay features had sustained approximately 6.1 million 18-kip [80-kN] ESALs in the outer lane and 1.2 million in the inner lane. All of the relief joints have reflected through the overlay and many of these reflective cracks have begun to spall, particularly in the section with the short relief joint spacing (40 ft [12 m]). It was also noted that there were more medium-severity transverse cracks in the vicinity of the relief joints and these cracks were wider than other cracks located further away from the relief joints. This is indicative of the movement of the underlying pavement into the relief joints.

The AC overlay above the relief joints was generally humped as a result of the expansion of the pavement into the relief joint area. Table 1 summarizes the measured relief joint humping. The three different sections (utilizing different relief joint spacings) each had different performance characteristics. The first section (1000-ft [305-m] relief joint spacing) had large "hump" measurements and slightly opened adjacent cracks. The second section (500-ft [152-m] relief joint spacing) had slightly smaller "hump" measurements, but adjacent cracks were wider and spalled slightly. Both of these sections were fairly rough riding. The last section (40-ft [12-m] relief joint spacing) was not humped and the ride was noticeably smoother. However, the transverse reflective cracks above the relief joints had spalled rather severely and there were many more deteriorated transverse cracks on the 40-foot [12 m] section. The control section, while not surveyed, was performing well in that the overlay was not extensively cracked and provided for a smooth ride.

Table 1. "Humping" of AC Overlay at Pressure Relief Joint Locations [1 in = 25.4 mm].

	INNER LANE			OUTER LANE	
SAMPLE UNIT	LEFT WHEELPATH HUMP (in)	RIGHT WHEELPATH HUMP (in)	CENTER LINE HUMP (in)	LEFT WHEELPATH HUMP (in)	RIGHT WHEELPATH HUMP (in)
01*	1.00	----	----	0.60	0.40
01	0.50	1.10	0.60	0.80	0.70
02	0.75	0.90	0.80	0.50	0.50
02	0.80	0.80	0.60	0.60	0.50
02	0.70	0.60	0.60	0.50	0.60
03	NO APPRECIABLE HUMPING OF PRESSURE RELIEF JOINTS				
Average	0.75	0.85	0.65	0.60	0.54
Std. Dev.	0.18	0.18	0.10	0.12	0.11

*This relief joint had been milled off once.

The longer-spaced pressure relief joints appear to have been somewhat appropriate for this project because they resulted in fewer deteriorated transverse cracks. However, the longer-spaced relief joints produced greater humping and a rougher ride. The control section did not develop any blowups or humping problems, although it was fairly short (one half mile [805 m]). The use of pressure relief joints resulted in varying degrees of pavement roughness due to humping, reflective cracking and deterioration of the overlay near the cracks. Deterioration of the overlay was most severe where the "D" cracking of the original pavement was not adequately addressed by either partial-depth repairs or full-depth asphalt concrete repairs (pressure relief joints) and had reflected through the overlay. Severe "D" cracking can be addressed only through the placement of full-depth PCC repairs or reconstruction. The placement of a number of full-depth repairs would have reduced any existing pressure buildup. Also, when "D" cracking exists at many joints it may have the effect of reducing pressure buildup since the strength of the concrete is greatly reduced at the joint areas.

Since the pavement was to be overlaid, it is likely that "D" cracking would have rapidly redeveloped outside of any full-depth PCC repairs and would have again caused the overlay to deteriorate. Thus, the asphalt concrete overlay of this severely "D" cracked pavement could be expected to provide good serviceability for only a short period of time, regardless of the pre-overlay rehabilitation.

The Indiana DOH has planned reconstruction for this project in 1986. This will consist of milling part of the existing AC overlay and placing a 10-in [254-mm] unbonded PCC overlay. This would have been a good alternative in 1975 as well.

OH071210 - The original pavement was 10-in [254-mm] JRCP with 60-ft [18.3-m] contraction joints and was constructed and opened to traffic in 1959. Two-foot [0.6-m] wide, skewed, asphalt concrete-filled pressure relief joints were installed in 1973 at half-mile [805 m] intervals in anticipation of pressure buildup problems. Full-depth repairs were placed at most joints at the same time to address severe "D" cracking. A 2.5-in [64-mm] asphalt concrete overlay was placed immediately afterward.

By 1985, the rehabilitation features had sustained approximately 12 million 18-kip [80-kN] ESALs in the outer lane and 3 million in the inner lane. All of the repair and relief joints have reflected through the overlay and many have deteriorated and have been repaired with temporary patch material. The AC overlay had been milled or planed above the relief joints to remove humps that had developed as the pavement expanded into the relief joint area. Figures 26 and 27 show typical relief joint conditions. Other areas of the overlay exhibited ravelling and weathering, up to 0.5 in [25 mm] of rutting, and low- and medium-severity potholes. A Roughness Index of 139 (fair) was obtained in the outer lane in 1984.

The pressure relief joints seem inappropriate for this project because there were no signs of pressure buildup problems and the installation of the full-depth repairs at every joint should have alleviated any built-up pressure.

Figure 26. Photo of AC Repairs Above Pressure Relief Joint and Regular Contraction Joint on OH071210.

Figure 27. Photo of Skewed Pressure Relief Joint on OH071210 (Same Relief Joint as Shown in Figure 26).

The installation of the pressure relief joints has increased the roughness of the overlay by allowing "humping" of the overlay above the pressure relief joints. Cracking and patching appears to have no correlation with the location of the pressure relief joints, although it is conceivable that the relief joints may have caused transverse joints and cracks to open wider than they otherwise would have, thus increasing both the severity and rate of development of reflective cracks. Condition survey data for this time period is unavailable to verify this theory. The overall life of this rehabilitation is expected to be 15 years.

Overlaid Relief Joints (Narrow--5 in [13 mm] wide or less)

KY065012 - The original pavement was 10-in [254 mm] JRCP with 50.0-ft [13.9 m] contraction joints and was constructed and opened to traffic in 1965 and 1966, respectively. Six-inch [152-mm] wide, skewed pressure relief joints were installed in 1982 at 1000-ft [305-m] intervals because blowups had previously occurred and additional pressure problems were anticipated. In addition, the original aggregate was known to be expansive. Six-inch [152-mm] perpendicular relief joints were also incorporated as approach or leave joints in some of the full-depth repairs that were placed as part of the pre-overlay repair. Approximately one third of the relief joints contained a preformed cellular plastic joint filler (ASTM D3204-80) and were covered with an 18-inch [457-mm] wide strip of commercial roofing paper prior to overlay. The remaining two thirds of the relief joints were filled with asphalt and these were the ones included in the condition surveys. Four-inch [100-mm] diameter perforated pipe underdrains were installed continuously along the project at the same time. A 4.0-in [100-mm] asphalt concrete overlay was then placed.

By 1985, the relief joints had sustained approximately 4.5 million 18-kip [80-kN] ESALs in the outer lane and 1.2 million in the inner lane. About one half of the original contraction joints had reflected through the overlay and few, if any, of the repair joints had reflected through. Rutting measurements in the outer lane averaged 0.13 in [3.3 mm].

The pressure relief joints have reflected through the overlay and the overlay is "humped" above the relief joints, although the hump over one of the relief joints has been milled off. It was noted that the hump between the wheel paths was generally larger than the hump within the wheel paths and that the hump measurements in the outer lane wheel paths were generally less than those in the inner lane. This indicates the response of the asphalt concrete mix to higher levels of traffic.

Only one of the three sample units surveyed appeared to indicate a relationship between pressure relief joint location and reflective crack location and severity.

The pressure relief joints were appropriate for this project in eliminating blowups since many blowups had occurred in the late 1960s and 1970s and were expected to continue due to the presence of expansive aggregate. The pavement is performing well, except for some roughness where the overlay has "humped." However, it is possible that improved joint maintenance practices and the construction of full-depth repairs at blowup locations would have provided equal or better service without the localized roughness and risk of more rapid joint and overlay deterioration that often accompanies pressure relief joints.

This pavement is expected to provide good serviceability for 4 to 8 years from the date of survey before rehabilitation is required. This provides a total life of 7 to 11 years. Periodic maintenance (relief joint milling and reflective crack sealing) are needed in the interim.

IL055252 - The original pavement was 10-in [254 mm] JRCP with 100-ft [31 m] contraction joints and was constructed and opened to traffic in 1956. Special "heavy-duty" expansion joints with dowels for load transfer were constructed in 1975 at 1500-ft [457-m] intervals because blowups had occurred previously and future pressure damage was anticipated. The relief joints are 6-foot [1.8-m] minimum length full-depth PCC repairs that are tied to the existing slab with deformed bars. Dowels measuring 1.25-in [32-mm] in diameter are incorporated across a 4-in [102 mm] formed joint which is filled with a 2-piece foam filler. The repair/relief joint is constructed slightly higher than the existing pavement so the new AC overlay can be constructed flush with the repair. The design of the relief joints placed on this rehabilitation project is given in Figure 19. A 3.5-in [90-mm] overlay binder course was placed in late 1975 and followed with a 1.5-in [38-mm] surface course in 1976.

By 1985, the pressure relief joints had sustained about 10 million 18-kip [80-kN] ESALs in the outer lane and 2.5 million in the inner lane. The original pavement had received more than 21 million ESALs in the outer lane and 5 million in the inner lane.

The condition surveys conducted in 1985 found low- and occasional medium-severity longitudinal cracking, reflection cracking, and rutting. Most of the cracks had been sealed. Rutting averaged 0.34 in [8.6 mm] in the outer lane and 0.17 in [4.3 mm] in the inner lane. A Roughness Index of 75 (smooth) was obtained with a BPR Roughometer in 1985. A skid number of 39 (marginal) was also measured in 1985.

The pressure relief joints have closed to an average width of 2.2 in [56 mm], have faulted an average of 0.08 in [2.0 mm] in the outer lane, and are considered acceptably smooth. The joint filler was generally still intact and keeping incompressibles from infiltrating, although in some instances the filler was partially absent. A slightly greater amount of cracking was observed near the surveyed pressure relief joints.

While the relief joints themselves are not extremely rough, a significant bump can be felt when driving across the joint. The surface of the overlay, which was once flush with the concrete surface, has moved downward. This is due to shoving and rutting of the overlay. While the faulting of these joints was not measured, the survey crew did note that they contributed significantly to the roughness of the pavement. There were no spalling problems or raveling problems at the relief joint-AC overlay interface (see Figures 28 and 29).

The "heavy-duty" pressure relief joints installed in 1975 have performed well in that they have withstood 10 years and approximately 10 million 18-kip [80-kN] ESALs without exhibiting excessive faulting or

Figure 28. Photo of Pressure Relief Joint Constructed on IL055252 (Sample Unit 1, Station 4+50).

Figure 29. Photo of Pressure Relief Joint Constructed on IL055252 (Sample Unit 2, Station 2+00).

significant spalling. There are some slight roughness problems on the approach and leave sides of the repairs, as described above. While this relief joint design is expensive, it does allow structural overlay of the pavement without the humping of the surface that often occurs when more traditional relief joint designs are used.

The need for pressure relief joints on this project is questionable. The relief joints have closed only 2.0 in [51 mm] in 10 years and the placement of full-depth repairs (as evidenced by the reflective cracking observed) probably relieved any pressure problems that existed at the time of overlay. The use of relief joints where they were not required may have resulted in premature reflection of the contraction and repair joints as they opened in response to the available expansion capacity of the pavement. In spite of this, the relief joints have performed very satisfactorily, and it cannot be determined that they have adversely affected the performance of the adjacent joints.

Overall, this rehabilitation project is considered successful, due to the good performance of the AC overlay. Considering the traffic and rate of deterioration of this pavement, the expected remaining life of this pavement is approximately 4-6 years from the survey date, when additional repairs and/or a structural overlay may be necessary. This would result in a total life of the rehabilitation of 13 to 15 years.

Relief Joints in Continuously Reinforced Concrete Pavement (CRCP)

OH070066 - The original pavement was 8-in [254-mm] CRCP and was constructed and opened to traffic in 1969. Four-inch [100-mm] wide, asphalt concrete-filled pressure relief joints were installed by maintenance forces in 1970 at half-mile [805-m] intervals in anticipation of pressure buildup problems. Two-foot [0.6-m] wide, skewed full-depth asphalt concrete repairs were placed in 1980 prior to placement of a 2.25 in [57 mm] asphalt concrete overlay, to address pavement roughness.

By 1985, the pressure relief joints had sustained about 21 million 18-kip [80-kN] ESALs in the outer lane and 5 million in the inner lane. The overlay had sustained about 7 million in the outer lane and 2.5 million in the inner lane. Some longitudinal and transverse reflection cracking was observed, but most of the cracks were sealed and had not spalled. Low-severity ravelling and weathering was observed at scattered locations throughout the surveyed sample units. Rutting measurements averaged 0.51 in [13 mm] in the outer lane and 0.15 in [3.8 mm] in the inner lane. A Roughness Index (GM Profilometer) of 84 (good) and a skid number (ASTM E274 - locked wheel) of 30 (marginal) were obtained in 1984. PSI was computed as 3.7 at this time.

The pressure relief joints have reflected through the overlay, have been sealed and are not spalled (see Figures 30 and 31). The overlay is not "humped" above the relief joints and the flexible repairs have not reflected through the overlay and have not caused "humping."

The interruption of the reinforcing steel in CRCP pavements is discouraged (unless the pavement has absolutely no remaining life and a structural overlay is to be placed) because the resulting loss of support

Figure 30. Photo of Sealed Crack Over Pressure Relief Joint on OH070066.

Figure 31. Photo of Sealed Crack Over Pressure Relief Joint on OH070066.

encourages rapid pavement deterioration in the area of the interruption. If such deterioration developed after the placement of pressure relief joints in 1970, prompting the placement of the 1980 overlay, it has not reflected through the overlay to date, nor have the full-depth bituminous repairs placed in 1980. The good performance of the overlay can be attributed to: (1) most of the relief joints had probably closed by the time the overlay was placed 10 years later, and (2) the overlay is apparently very pliable, as indicated by the relatively low occurrence of reflective cracks, the high rutting measurements, and the low skid number.

Although the use of pressure relief joints and full-depth bituminous repairs has apparently not adversely affected the performance of this project, their use was probably not justified. It would seem that improved joint maintenance practices and the construction of full-depth repairs at blowup locations would have provided equal or better service without the localized roughness and risk of more rapid joint and overlay deterioration that often accompanies pressure relief joints.

This rehabilitation project is considered successful because of the good performance of the AC overlay, although rutting and pavement friction are becoming a problem. This pavement is expected to provide good serviceability for 4-6 years (6.4 to 9.6 million outer lane ESALs) from the date of survey before major rehabilitation (structural overlay) is required. This would provide a total life of the rehabilitation of 9 to 11 years. ODOT plans to replace the wheel saw cut pressure relief joints with dowelled and undercut joint repairs in 1986.

OH270029 - The original pavement was six lanes of 8-in [254-mm] CRCP and was constructed and opened to traffic in 1968 and 1969, respectively. Four-inch [100-mm] wide, asphalt concrete-filled pressure relief joints were installed in 1983 at 2000-ft [610-m] intervals due to perceived pressure problems and fear of accident liability (from blowups). No other rehabilitation was undertaken.

By 1985, the pressure relief joints had sustained about 1 million 18-kip [80-kN] ESALs in the outer lane, 0.4 million in the middle lane and 0.1 million in the inner lane. The original pavement had sustained about 5 million in the outer lane, 2.5 million in the middle lane and 0.6 million in the inner lane.

A few edge punchouts and deteriorated transverse cracks were observed in the original pavement. The keyway in the longitudinal joint between the outer and middle lanes had been removed and this joint had been repaired with asphalt concrete. A Roughness Index (GM Profilometer) of 112 (fair) and a skid number (ASTM E274 - locked wheel) of 31 (marginal) were obtained in 1984. A Present Serviceability Index of 3.0 was computed at this time.

The pressure relief joints are exhibiting medium-severity joint and corner spalling and transverse cracks adjacent to the pressure relief joints have deteriorated from tight, nonworking cracks (typical of a CRC pavement) to spalled, working cracks (see Figures 32 and 33). Punchouts were more frequently located near the pressure relief joints.

Figure 32. Photo of Pressure Relief Joint on OH270029.

Figure 33. Photo of Pressure Relief Joint on OH270029 (Same Relief Joint as in Figure 32).

Installation of pressure relief joints on this project was inappropriate because the compressive stresses in areas that had suffered blowups in the past were probably sufficiently relieved by the blowups. If the CRCP is repaired soon after punchouts or ruptured steel occurs there appears no reason for a buildup of stresses because the tight cracks do not allow for the infiltration of incompressibles. Further, the interruption of the continuous reinforcing steel resulted in premature deterioration of adjacent transverse cracks. If it was absolutely necessary to relieve pressure, a better approach would have been to remove pavement sections for the placement of traditional CRC repairs and leave these areas open for 24-72 hours. A properly constructed repair could then have been installed by carrying the steel through the repair to keep the cracks tight. This would also have reduced the possibility of loss of support caused by allowing water to enter a wide relief joint.

This rehabilitation project is not considered successful because of the poor performance of the pavement in the areas of the pressure relief joints. The relief joints have deteriorated under relatively light traffic (only 7% trucks) and rehabilitation of these areas is needed now.

The performance of the pressure relief joints in this project (OH270029) was similar to that of the project adjacent to it (OH270031). The pavements are similar in all respects except for the reinforcement (OH270029 had larger rebar) and the base (OH270029 used 3 in [76 mm] of a bituminous aggregate mixture, while OH270031 used 4 in [102 mm] of cement-treated base). The poor performance of the relief joints on both projects is due to the pavement continuity being destroyed.

OH270031 - The original pavement was six lanes of 8-in [254-mm] CRCP and was constructed and opened to traffic in 1968 and 1969, respectively. Four-inch [100-mm] wide, asphalt concrete-filled pressure relief joints were installed in 1983 at 2000-ft [610-m] intervals because of perceived pressure problems and fear of accident liability (from blowups). No other rehabilitation was undertaken.

By 1985, the pressure relief joints had sustained about 1 million 18-kip [80-kN] ESALs in the outer lane, 0.4 million in the middle lane and 0.1 million in the inner lane. The original pavement had sustained about 5 million in the outer lane, 2.5 million in the middle lane and 0.6 million in the inner lane. Several edge punchouts were observed, particularly in the inner lane. Transverse cracking was also slightly more severe and more concentrated in the inner lane, where deteriorated cracks were observed at an average spacing of 31 ft [9.5 m]. The keyway in the longitudinal joint between the outer and middle lanes had recently failed (due to spalling problems) and localized temporary asphalt concrete repairs had been placed in these areas. A Roughness Index (GM Profilometer) of 112 (fair) and a skid number (ASTM E274 - locked wheel) of 31 (marginal) were obtained in 1984. A Present Serviceability Index of 3.0 was computed at this time.

The pressure relief joints are exhibiting medium-severity joint and corner spalling. In addition, transverse cracks adjacent to the pressure relief joints have deteriorated from tight, nonworking cracks (typical of a CRC pavement) to spalled, working cracks (see Figures 34 and 35). Punchouts were more frequently located near the pressure relief joints.

Figure 34. Photo of Pressure Relief Joint on OH270031.

Figure 35. Photo of Pressure Relief Joint on OH270031.

Installation of pressure relief joints on this project was inappropriate because the compressive stresses in areas that had suffered blowups in the past were probably sufficiently relieved by the blowups. Further, the interruption of the continuous reinforcing steel resulted in premature deterioration of adjacent transverse cracks. A better approach might have been to remove pavement sections for the placement of traditional CRC repairs and leave these areas open for 24-72 hours to allow for pressure relief. A properly constructed repair could then have been installed by carrying the steel through the repair to keep the cracks tight. This would have reduced the possibility of loss of support caused by allowing water to enter a wide relief joint.

This rehabilitation project is not considered successful because of the poor performance of the pavement in the areas of the pressure relief joints. The relief joints have deteriorated under relatively light traffic (only 7% trucks) and rehabilitation of these areas is needed now.

IL072067 - The original pavement was 7-in [178 mm] CRCP constructed over 4-in [102 mm] of bituminous stabilized aggregate subbase and opened to traffic in 1970. The CRCP had developed severe "D" cracking deterioration and was spalling in the wheel paths. Four-foot [1.2-m] wide, asphalt concrete-filled pressure relief joints were installed in 1983 at intervals ranging from 1000 ft [305 m] to 1300 ft [396 m] in anticipation of pressure buildup problems. A 2.75 in [70 mm] asphalt concrete overlay was placed immediately afterward.

By 1986, the rehabilitation features had sustained approximately 1.8 million 18-kip [80-kN] ESALs in the outer lane and 0.3 million in the inner lane. The original pavement has sustained approximately 7 million 18-kip [80-kN] ESALs in the outer lane and 1 million in the inner lane.

Low-severity, unsealed transverse cracks were found throughout the project, but were concentrated in the first sample unit. Rutting averaged less than 0.1 in [2.5 mm] in the outer lane and was negligible in the inner lane. A Roughness Index of 70 (smooth) was obtained in 1985 with a BPR Roughometer. Surface friction was also measured in 1985 using a locked-wheel trailer and a friction number of 51 (good) was obtained.

Medium-severity, unsealed transverse reflection cracks were generally associated with all of the relief joints surveyed (see Figures 36 and 37). The portion of the AC overlay directly above the relief joints was often "humped" from the movement of the CRCP into the relief joint. Higher volumes of traffic appear to push the AC from the wheel paths toward the centers and edges of the lanes. This was not as apparent in the inner lane, where truck volumes are lower. There was no apparent correlation between the location and severity of transverse cracks and relief joint location, although it was noted that the sections of the pavement with the highest density and severity of transverse cracking did not suffer from "humping" at the relief joints. Table 2 summarizes the measurement of "humping" at various locations on this project.

The pressure relief joints placed on this project are inappropriate because there are no signs that pressure buildup problems existed. Furthermore, the interruption of the continuous reinforcement can be

Figure 36. Photo of AC Overlay Above Pressure Relief Joint in Sample Unit 1 (Station 1+84) of IL072067.

Figure 37. Photo of AC Overlay Above Pressure Relief Joint in Sample Unit 4 (Station 3+75) of IL072067.

Table 2. "Humping" of AC Overlay at Pressure Relief Joint Locations [1 in = 25.4 mm].

	INNER LANE			OUTER LANE		
	LEFT WHEELPATH	CENTER OF LANE	RIGHT WHEELPATH	LEFT WHEELPATH	CENTER OF LANE	RIGHT WHEELPATH
Sample Unit	Hump(in)	Hump(in)	Hump(in)	Hump(in)	Hump(in)	Hump(in)
1	N O	H U M P I N G		N O	H U M P I N G	
2	N O	H U M P I N G		N O	H U M P I N G	
3	0.40	0.90	0.70	0.40	0.60	0.10
3	0.30	0.40	0.40	0.40	0.50	0.30
4	0.30	0.40	0.50	0.60	0.70	0.30
4	0.20	0.50	0.40	0.30	0.70	0.30
AVG.	0.30	0.55	0.50	0.43	0.63	0.25
STD. DEV.	0.08	0.24	0.14	0.13	0.10	0.10

expected to result in accelerated deterioration of the transverse cracks in the immediate vicinity of the relief joints.

The relief joints seem to have performed marginally well with the asphalt concrete overlay in three years of service. The "humping" presents a slight roughness problem now, although it is not reflected by the roughness index.

This rehabilitation project is considered unsuccessful because the AC overlay is not performing well in the area of the relief joints. Considering current pavement conditions and traffic levels, it is anticipated that major rehabilitation will be required in 4-6 years, although interim maintenance (crack sealing, relief joint milling, etc.) is likely to be required. This provides a total life of 7 to 9 years. The relief joints should be replaced with traditional CRC repairs to restore continuity.

Narrow Relief Joints (5 in [13 mm] wide or less)

IA030156 - The original pavement was 10-in [254 mm] JPCP with 20-ft [6-m] contraction joints and was constructed and opened to traffic in 1964. Four-inch [102-mm] wide, polyurethane foam-filled (Flex-Loc) pressure relief joints were installed in 1980 using Vermeer wheel saws at 1000-ft [305-m] intervals to prevent pressure buildup problems and blowups. Details concerning the sealant cap that was placed are not available. Additional relief joints were placed near bridges. By 1985, the pressure relief joints had sustained about 0.6 million 18-kip [80-kN] ESALs and the original pavement had received more than 2 million.

A joint resealing program was conducted within the project in 1984. The joints were refaced and W. R. Meadows "SofSeal," a hot-poured sealant, was installed over a backer rod. Design and performance details for the joint resealing program are described elsewhere in this report.

The condition surveys conducted in 1985 revealed low-severity longitudinal cracks extending 5 to 10 ft [1.5-3 m] on either side of several transverse joints (see Figure 38). It is believed that these are compression cracks caused by entrapment of a few large incompressibles deep in the joints and the subsequent buildup of compressive stresses during periods of pavement expansion. Transverse cracks were found only where transverse joints were omitted and 40-ft [12-m] panels were present. At these locations a medium-severity transverse crack developed near the center of the panel. One corner break was observed adjacent to a pressure relief joint. Low-severity "D" cracking was found occasionally and had often resulted in low-severity transverse joint or corner spalling. Most of the original pavement joints were acceptably smooth, with an average joint faulting measurement of 0.11 in [2.8 mm]. A Present Serviceability Index of Roughness (PSIR) of 3.23 (fair) was obtained using an IJK Roadmeter in 1983 (corresponding to a BPR Roughometer Roughness Index of 124). An average skid number of 33 (marginal) was obtained with a locked-wheel skid trailer in 1984.

The pressure relief joints (excluding those located near bridges) have closed to an average width of 0.54 in [14 mm], which is about the same width as the resealed joints. The relief joints have faulted an average of 0.16 in [4.1 mm] after only 0.6 million ESALs, which compares with 0.11 in [2.8 mm] and 2.2 million ESALs for the original contraction joints. Half of the surveyed relief joints had no filler present and incompressibles were observed in all of the surveyed relief joints. Several of the relief joints also exhibited medium-severity transverse joint spalling (see Figures 39 and 40).

The pressure relief joints installed in 1980 were inappropriate for this project. They were placed to relieve pressure and prevent blowups and other pressure-related damage. However, since no pressure damage had previously occurred and the pavement has short joint spacing, it is doubtful that the a potential for pressure damage existed. It is unclear whether compressive stresses were building (e.g., did the observed longitudinal compression cracks form before or after 1980?) and, if so, whether the installation of pressure relief joints was the best approach to address the problem.

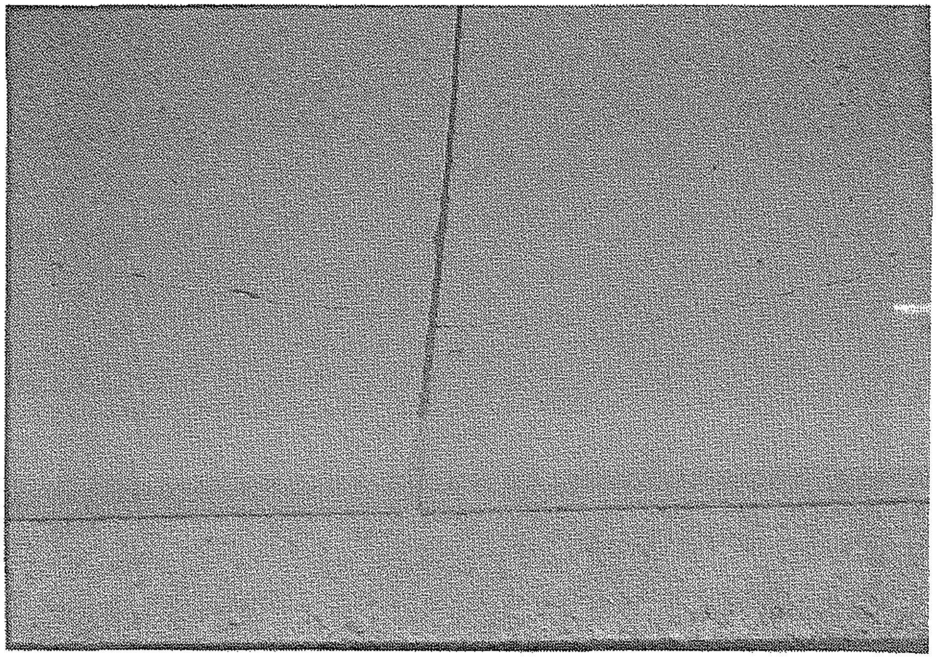


Figure 38. Photo of Regular Contraction Joint with Longitudinal Compression Cracks on IA030156.

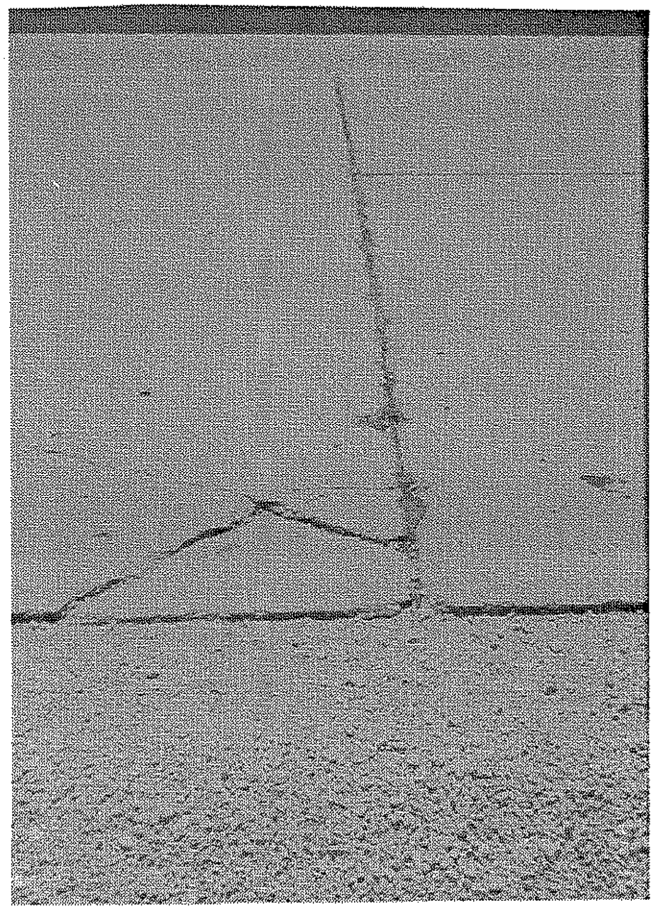


Figure 39. Photo of Typical Pressure Relief Joint in Sample Unit 2 (Station 8+60) on IA030156.

Figure 40. Photo of Typical Pressure Relief Joint in Sample Unit 1 (Station 11+40) on IA030156.

The presence of the pressure relief joints contributed to further opening of the transverse contraction joints, enabling water and additional incompressibles to enter the joints. This opening would also reduce the effectiveness of the aggregate interlock load transfer, enabling pumping and increased faulting to develop. The Iowa Department of Transportation has recently changed their pressure relief joint policy and now limits their use, particularly on JPCP.

This rehabilitation project is considered successful, although it is believed that improved performance could have been obtained by eliminating the pressure relief joints and following a more rigorous joint resealing and maintenance program.

Considering the current pavement condition, rate of deterioration, and current accumulation of traffic, the pavement can be expected to provide acceptable performance for approximately 8 to 10 years from the date of the survey. Thus, major rehabilitation, in the form of diamond grinding and joint repairs, is expected as early as 1993 or as late as 1995 for a total life since rehabilitation of 13 to 15 years. Local rehabilitation is presently needed near some of the relief joints.

IA035086 - The original pavement was 10-in [254 mm] JRCP with 76.5-ft [23.3 m] contraction joints and was constructed and opened to traffic in 1965. Four-inch [102-mm] wide, polyurethane foam-filled (Flex-Loc) pressure relief joints were installed in 1980 using Vermeer wheel saws at 1000-ft [305-m] intervals to prevent pressure buildup problems and blowups. Details concerning the sealant cap that was placed are not available. Additional relief joints were placed near bridges.

Full-depth repairs have been placed at about 70% of the original contraction joints, with placement beginning in 1977 and continuing to the present. They have been placed to address transverse joint deterioration (generally due to "D" cracking) and represent the evolution of full-depth repair design in Iowa since 1977. Their designs and performance are detailed in other portions of this report.

By 1985, the pressure relief joints had sustained about 2.5 million 18-kip [80-kN] ESALs in the outer lane and about 0.5 million in the inner lane. The original pavement had received more than 7 million in the outer lane and 1.3 million in the inner lane.

The condition surveys conducted in 1985 revealed low- and medium-severity transverse cracking (average crack spacing of 25 ft [7.5 m] in the outer lane), joint spalling, "D" cracking, localized areas of scaling, joint sealant deterioration and unacceptable joint faulting, especially at the repair and relief joints. A Present Serviceability Index of Roughness (PSIR) of 3.75 (fair-good) was obtained using a IJK Roadmeter in 1983. The corresponding BPR Roughometer Roughness Index was 90. A skid number of 37 (marginal) was obtained with a locked wheel skid trailer in 1984.

The pressure relief joints (excluding those located near bridges) have closed to an average width of 0.84 in [21 mm] and have faulted an average of 0.23 in [5.7 mm]. The filler is frequently absent and the joints are full of incompressibles (see Figures 41 and 42). Several of the relief

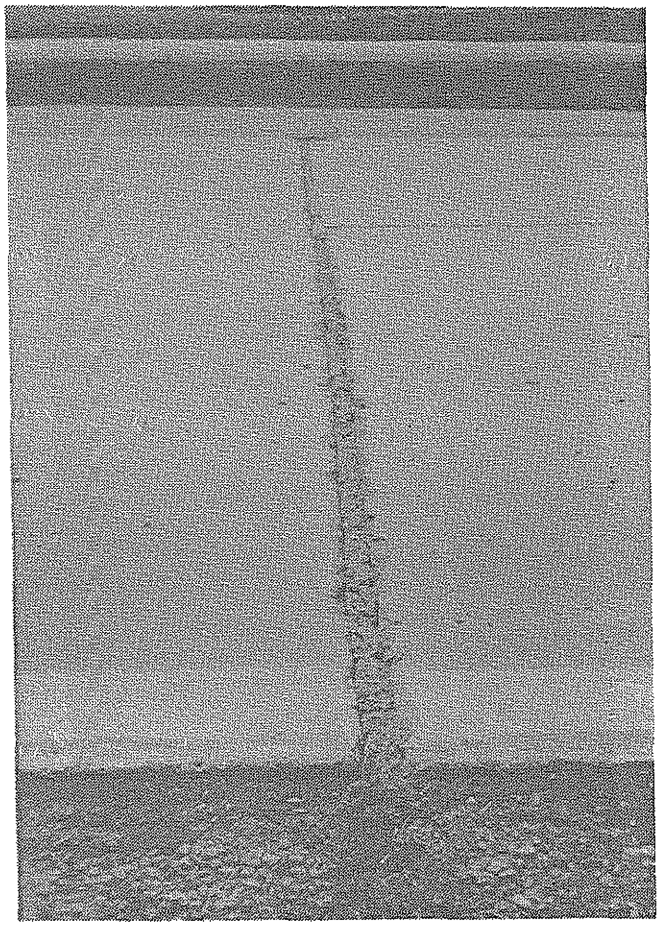


Figure 41. Photo of Pressure Relief Joint
Filled with Incompressibles on IA035086.

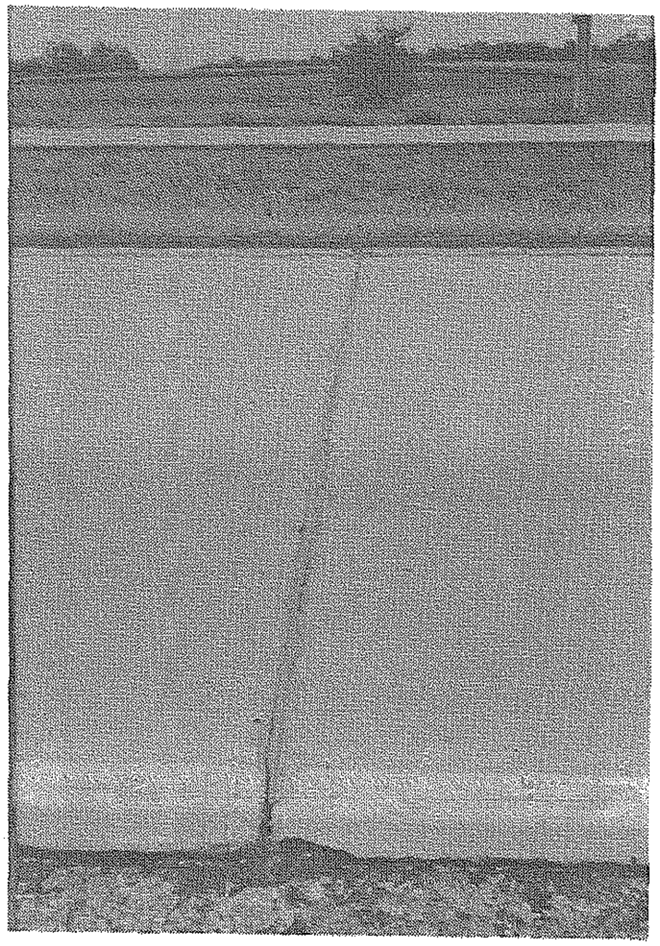


Figure 42. Photo of Pressure Relief Joint with Filler Intact on IA035086.

joints have medium-severity spalls along the transverse joints and at the corners. Most of the adjacent original contraction joints have been replaced, but those that are present are faulted nearly twice as much as those located at some distance from the relief joints.

The pressure relief joints were inappropriate for this project because no evidence of blowups occurring prior to installation of the relief joints was observed and the placement of repairs in every year since 1977 would have relieved much of the compressive stress present. Although most of the original contraction joints have been replaced with full-depth repairs, faulting, spalling and joint width measurements of the remaining joints indicate the adverse effects of unnecessary relief (see Table 3). Also, some of the spalling and faulting of the 1979 and 1981 undowelled repairs may be due to the opening of their joints (due to the presence of the relief joints and other repair openings) which allowed the entry of incompressibles and later produced spalling.

This rehabilitation project is considered successful because the overall serviceability of the pavement has been maintained by the replacement of joints and repair of deteriorated cracks as necessary. The superior performance of the more recent repair designs is documented in another portion of this report. It is believed that the pre-1983 repairs (undowelled) will continue to deteriorate rapidly and require rehabilitation within the next few years to maintain serviceability.

It was noted that the best performance was found in a section of pavement constructed with an aggregate less susceptible to "D" cracking. This section exhibits little distress and clearly indicates that "D" cracking (i.e., a nondurable aggregate) was the major contributing factor to the deterioration of the rest of the project.

IL055098 - The original pavement was 10-in [254 mm] JRCP with 100-ft [30.5 m] contraction joints and was constructed and opened to traffic in 1962 and 1963, respectively. Four-inch [100-mm] wide, ethafoam-filled pressure relief joints were installed in 1970-1975 using Vermeer wheel saws at quarter-mile [402-m] intervals as part of a "policy" aimed at preventing pressure buildup problems. The relief joints were sealed with asphalt sealant material. Additional relief joints were placed near bridges in 1983. Full-depth repairs were placed at most joints in 1983 to address a moderate-to-severe "D" cracking problem. About half of these repairs were replaced in 1984 due to deep spalling and loss of load transfer. All of the repairs utilized three 1.25-in [32-mm] dowels in the outer wheel path and two in the inner wheel path for load transfer. Some of the 1984 repairs included an extra dowel in each wheel path and a built-in two-inch [51-mm] wide relief joint filled with Ceramar preformed plastic joint filler and a hot-poured joint sealant conforming to ASTM D3405. Four-inch [100-mm] diameter slotted plastic pipe drains were placed continuously along the project after the full-depth repairs were in place in 1983. The approach and leave slabs surrounding each repair were subsealed with cement grout in 1984 after the failed 1983 repairs were replaced.

By 1985, the pressure relief joints had sustained between 9 and 13 million 18-kip [80-kN] ESALs in the outer lane and between 2 and three million ESALs in the inner lane. The other techniques had sustained about

Table 3. Summary of Joint Faults and Widths in Sample Unit 2 (Milepost 105.0-Southbound) of IA035086 [1 in = 25.4 mm].

OUTER				INNER			
STATION	TYPE OF JOINT	FAULT (in.)	WIDTH (in.)	STATION	TYPE OF JOINT	FAULT (in.)	WIDTH (in.)
0+00	Reg Cont	0.13	0.7	0+00	Reg Cont	0.05	0.6
0+76	Reg Cont	0.09	0.5	0+76	Reg Cont	0.05	0.6
1+53	Reg Cont	0.08	0.7	1+53	Reg Cont	0.00	0.7
2+29	Reg Cont	0.17	0.4	2+29	Reg Cont	0.03	0.7
3+06	Reg Cont	0.17	0.5	3+06	Reg Cont	0.00	0.6
3+82	Reg Cont	0.06	0.5	3+82	Reg Cont	-0.01	0.6
4+59	Reg Cont	0.12	0.6	4+59	Reg Cont	0.05	0.7
5+35	Reg Cont	0.25	0.5	5+35	Reg Cont	0.04	0.6
6+12	Reg Cont	0.22	0.5	6+12	Reg Cont	0.04	0.7
6+47	Pr Relief	0.32	0.9	6+47	Pr Relief	0.09	0.7
6+88	Reg Cont	0.33	0.5	6+88	Reg Cont	0.03	0.5
7+65	Reg Cont	0.35	0.7	7+65	Reg Cont	0.06	0.6
8+41	Reg Cont	0.15	0.5	8+41	Reg Cont	0.02	0.6
9+17	Reg Cont	0.14	0.6	9+17	Reg Cont	0.01	0.6
9+93	Reg Cont	0.10	0.5	9+93	Reg Cont	0.03	0.5
10+70	Reg Cont	0.07	0.5	10+70	Reg Cont	-0.02	0.5

Note: Positive faulting measurements indicate that the approach side of the joint is higher than the leave side. Negative faulting measurements indicate that the leave side of the joint is higher than the approach side.

2 million 18-kip [80-kN] ESALs in the outer lane and 0.5 million in the inner lane and the original pavement had received more than 17 million in the outer lane and 3 million in the inner lane.

The condition surveys conducted in 1985 revealed low- and occasional medium-severity transverse cracking, joint and corner spalls, "D" cracking, localized areas of scaling, joint sealant deterioration and unacceptable joint faulting. Many of the drain outlets were partially filled with silt and sediment. A Roughness Index of 114 (slightly rough) was obtained with a BPR Roughometer in 1985. A skid number of 50 (good) was obtained with a locked-wheel skid trailer in 1985.

The pressure relief joints that were installed in the early 1970s have closed to an average width of 0.8 in [20 mm] and have faulted an average of 0.22 in [5.6 mm]. The filler is intact, but the sealant cap is missing. The adjacent original contraction joints have opened and are filled with incompressibles. Where relief joints were incorporated as full-depth repair joints, they have not closed appreciably and are still sealed. None of the relief joints have exhibited significant transverse joint spalling, although corner spalling was observed near the outer shoulder. The full-depth repair performance is described elsewhere in this report.

The pressure relief joints were inappropriate for this project because there were no signs of pressure buildup problems and the installation of the full-depth repairs at every joint should have alleviated any built-up pressure. The installation of these joints may have aggravated the pumping and loss of load transfer problems of the pavement, as seen by the spalling and faulting of the repairs soon after their placement. Also, the adjacent original transverse contraction joints and working cracks near the pressure relief joints have opened and filled with incompressibles.

Although the expected remaining life before rehabilitation of this pavement is approximately three to five years from the date of survey, IDOT planned an asphalt concrete overlay for the fall of 1985 to improve the rideability of the pavement.

IL055102 - The original pavement was 10-in [254 mm] JRCP with 100 ft [30.5 m] contraction joints and was constructed in 1962 and opened to traffic in 1963. Four-inch [100-mm] wide, ethafoam-filled pressure relief joints were installed in the period of 1970-1975 using Vermeer wheel saws at quarter-mile [402-m] intervals as part of a "policy" aimed at preventing pressure buildup problems. The relief joints were sealed with a hot-poured asphalt sealant. Relief joints were also placed in 1983 near bridges.

Full-depth repairs were placed at about 20% of the joints in 1983 to address joint and corner spalling (not related to "D" cracking) and corner breaks. These were generally partial-lane width repairs utilizing three 1.25-in [32-mm] dowels in the outer wheel path and two in the inner wheel path for load transfer. Four-inch [100-mm] diameter slotted plastic pipe drains were placed continuously along the project after the full-depth repairs were in place in 1983. Portions of the surveyed pavement were part of an experimental project on undersealing (66). Diamond grinding was performed in 1983 after the installation of the full-depth repairs.

By 1985, the pressure relief joints had sustained between 7 and 11 million 18-kip [80-kN] ESALs in the outer lane and between 1.5 and 2 million in the inner lane. The other techniques had sustained about 2 million 18-kip [80-kN] ESALs in the outer lane and 0.5 million in the inner lane and the original pavement had received more than 14 million in the outer lane and 2 million in the inner lane.

The condition surveys conducted in 1985 showed joint sealant deterioration, incompressible-filled transverse joints, medium-severity transverse joint and corner spalling, low- and medium-severity transverse cracking (20-ft [6-m] intervals) and localized areas of scaling. The remaining transverse contraction joints were faulted an average of 0.03 in [0.7 mm] and were open an average of 0.4 in [10 mm]. Many of the drain outlets were partially filled with silt and sediment. A Roughness Index of 66 (very smooth) was obtained with a BPR Roughometer in 1985. A skid number of 49 (good) was obtained with a locked wheel skid trailer in 1985.

The pressure relief joints that were installed in the early 1970s have closed to an average width of 1.0 in [25 mm] and have faulted an average of 0.06 in [1.5 mm]. The filler and sealant are absent from some of the surveyed relief joints and these joints are filled with incompressibles. The adjacent original contraction joints are opened slightly more than the average width and are also filled with incompressibles. Some of the relief joints have exhibited low- to medium-severity transverse joint spalling and some corner spalling (see Figures 43 and 44). The full-depth repair performance is described and discussed in another portion of this report.

The pressure relief joints were inappropriate for this project because there were no signs of pressure buildup problems. Their use has had the undesirable effect of allowing many of the remaining original contraction joints to open and fill with incompressibles. These open adjacent joints can be expected to fault more rapidly as water enters and causes pumping of the dense-graded subbase. The relief joints themselves are already showing much more faulting than the average contraction joints.

This project is considered successful largely because of the smooth surface provided by the diamond grinding operation. The expected remaining life before rehabilitation of this pavement is approximately four to seven years from the date of survey, although it could be serviceable for a longer period if the experimental undersealing project successfully reduces the pumping and faulting.

This project is very similar to the adjacent project, IL055098, but has performed much better primarily because of the inclusion of a non-"D" cracking aggregate in the original construction project. It has also received slightly less traffic.

IL080105 - The original pavement was a 10-in [254 mm] JRCP with 100-ft [30.5 m] contraction joints and was constructed and opened to traffic in 1960. Four-inch [100-mm] wide, cellular plastic-filled pressure relief joints were installed near midslab at quarter-mile [402-m] intervals in 1984 using wheel saws in anticipation of pressure buildup problems. A few of the relief joints were filled with asphalt concrete instead of preformed cellular plastic filler. The relief joints were sealed with a

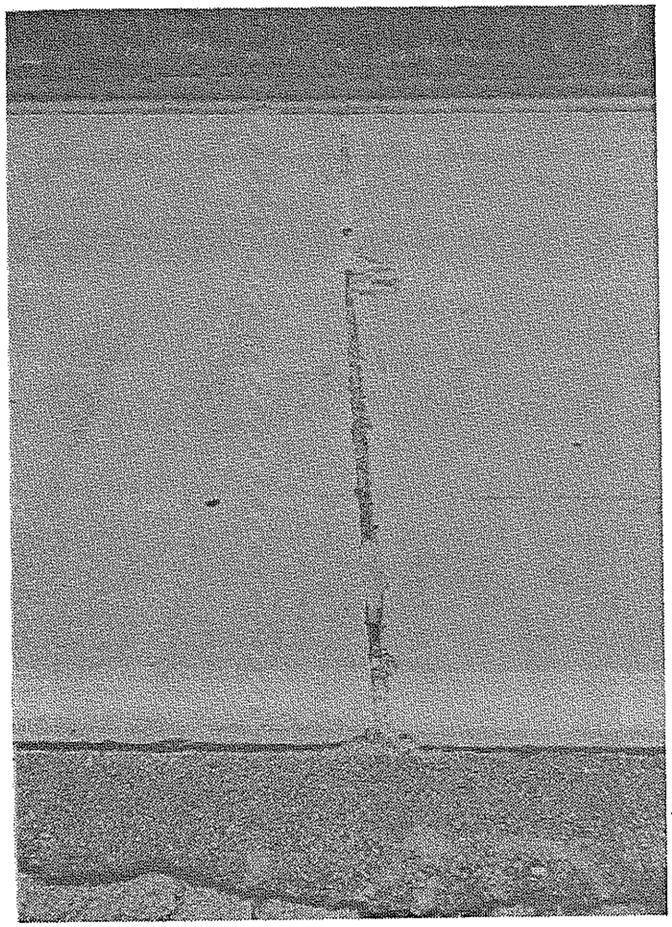


Figure 43. Photo of Pressure Joint on IL055102 with Medium- to High-Severity Spalling.

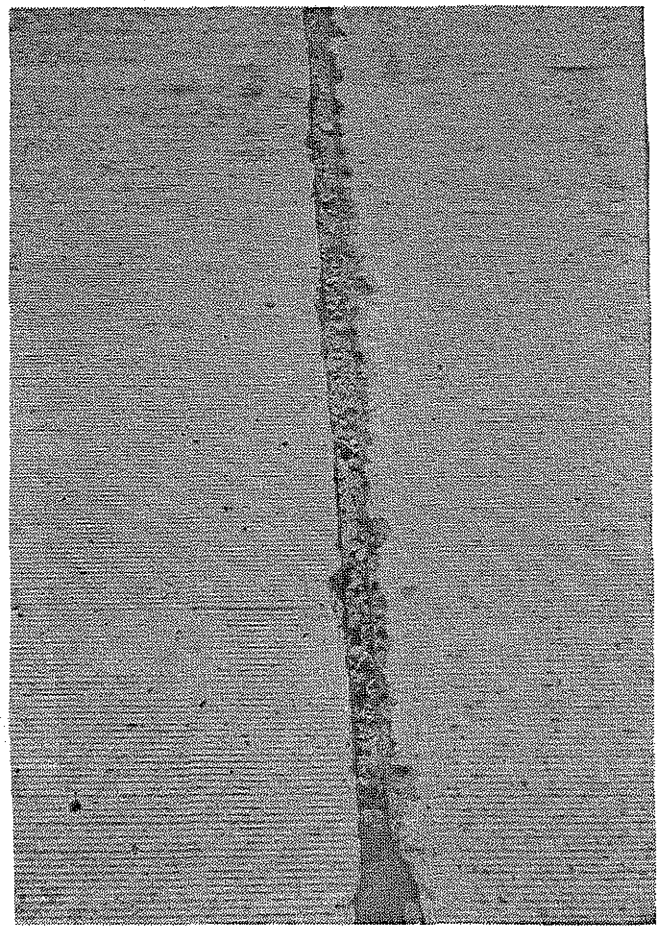


Figure 44. Photo of Pressure Relief Joint Filled with Incompressibles on IL055102.

cold-applied asphalt sealant material meeting ASTM D1850. Additional rehabilitation techniques applied include full-depth repairs (1983 and 1985), an experimental installation of longitudinal underdrains (1983), cement-grout undersealing (1984), diamond grinding (1984), load transfer restoration (1984), and joint resealing (1984). The design and performance of these additional rehabilitation techniques are described in detail in Reference 66. The design and performance of the full-depth repairs and joint resealing projects are also described in other portions of this report.

By 1985, the pressure relief joints had sustained about 1 million 18-kip [80 kN] ESALs in the outer lane and 0.2 million in the inner lane. The original pavement had received nearly 16 million in the outer lane and 3 million in the inner lane.

The condition surveys conducted in 1985 found a lot of medium-severity transverse cracks (many containing retrofit load transfer devices) and some low- and high-severity cracks. The average crack spacing was 15-20 ft [4.5-6.0 m]. Medium-severity longitudinal joint spalling was found throughout two of the four sample units surveyed, although "D" cracking and reactive aggregate were not observed.

Joint sealant was absent from more than 50% of the joints that were not included in the 1984 resealing project and incompressibles were observed in 7% of these joints. All of these original contraction joints exhibited low- or medium-severity transverse joint and corner spalling. The joints included in the resealing program were all well-sealed and unspalled. The remaining transverse contraction joints were faulted an average of 0.03 in [0.7 mm] in 1985 and an average of 0.07 in [1.8 mm] in 1986. There was a significant difference in faulting between sections with different drainage designs.

A history of average roughness index and surface friction values for the project is presented in the following table:

YEAR	ROUGHNESS INDEX	SURFACE FRICTION
1980	- - -	36 (marginal)
1981	- - -	- - -
1982	151 (rough)	- - -
1983*	51 (very smooth)	48 (good)
1984	73 (very smooth)	40 (good)
1985	64 (very smooth)	- - -
1986	82 (smooth)	40 (good)

*Diamond grinding was performed this year.

Roughness was measured using a BPR Roughometer, while surface friction was measured using a locked-wheel trailer.

The pressure relief joints that were installed in 1984 had closed to an average width of 3.5 in [89 mm] and had faulted an average of 0.16 in [4.0 mm] in 1985 (twice the faulting of the original contraction joints). The filler was absent from two of the three relief joints surveyed and these joints contained incompressibles (see Figures 45 and 46). The third relief joint was located in the joint resealing project and it showed no

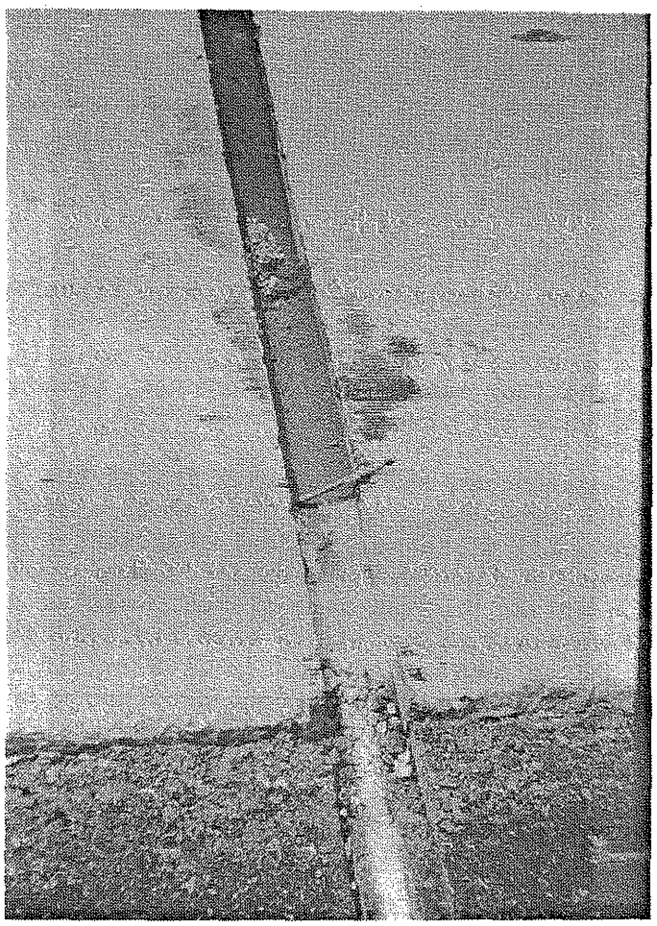


Figure 45. Photo of Typical Pressure Relief Joint in Sample Unit 1 (Station 2+40) of IL080105.

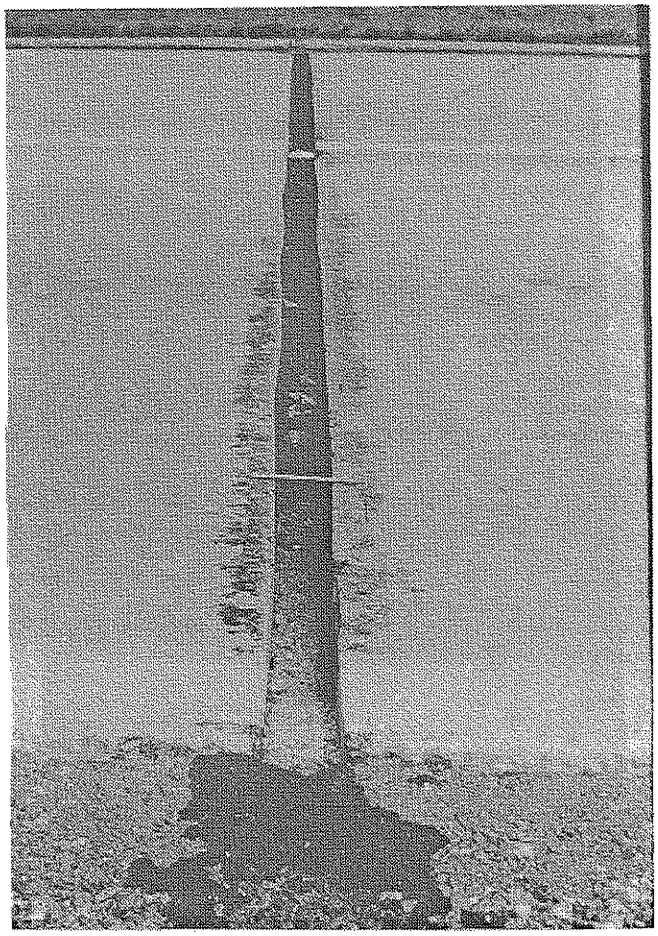


Figure 46. Photo of Pressure Relief Joint Filled with AC in Sample Unit 2 (Station 7+30) of IL080105.

signs of deterioration. Adjacent contraction joint faulting on the approach side of the relief joints was about the same as the average faulting for the project; adjacent contraction joint faulting on the leave side of the relief joints was slightly higher. Joint widths surrounding the relief joints were not significantly wider than the project average.

The pressure relief joints were inappropriate for this project because there are no signs that pressure buildup problems existed at the time of their placement, as indicated by the very small relief joint closure that has taken place. The placement of full-depth repairs in 1983 and again in 1985 would also have relieved much of any pressure buildup problem, making the installation of relief joints unnecessary The presence of additional expansion capacity probably contributed to (but was not by any means solely responsible for) the premature failure of the experimental load transfer devices that were installed on this project, as well as the more rapid deterioration of transverse cracks in the area of the relief joints.

Overall, this rehabilitation project is considered moderately successful. Good performance was obtained from the full-depth repairs, joint resealing, diamond grinding, and longitudinal underdrains, but there were also problems with the load transfer restoration rehabilitation.

While major rehabilitation will clearly be required in several years, some activities should be undertaken now to maintain a higher overall level of serviceability:

1. A review of the 1983 repairs should be conducted, with removal and replacement of those exhibiting serious distress.

2. The failed load transfer devices should be removed and replaced with more reliable devices.

3. A joint resealing project should be performed over the entire project, although there is a problem of sealing 100 ft [30.5 m] slabs.

4. A crack sealing program should be performed.

LA055032 - The original pavement was 10 in [254 mm] JRCP with 58.5-ft [17.8-m] contraction joints and was constructed and opened to traffic in 1966. Four-inch [102-mm] wide, cellular plastic-filled (Flex-Loc) pressure relief joints were installed in the northbound lanes in 1980 using Vermeer wheel saws at half-mile [805-m] intervals to relieve expansive pressures and, thus, eliminate blowups. The southbound lanes served as a "control" section, where no relief joints were placed. This experimental project evaluated the performance of selected relief joints and the economic benefits of cutting such relief joints versus not cutting the joints and the associated cost of repairing resulting blowups (99).

By 1985, the pressure relief joints had sustained about 2.7 million 18-kip [80-kN] ESALs in the outer lane and 0.5 million in the inner lane while the original pavement had received more than 5.5 million in the outer lane and 0.8 million in the inner lane.

The condition surveys conducted in 1985 revealed some low-severity transverse slab cracking in both directions and one full-depth repair in the southbound lanes where a blowup had occurred at a joint. A total of 15 blowups occurred in the southbound lane over the 25-mile [33.8-km] section while none occurred in the northbound lane where the pressure relief joints were placed (99).

The joint sealant was absent from nearly 20% of the surveyed original transverse contraction joints and varying amounts of incompressibles were observed in most of the joints throughout the project. Forty-five percent of the southbound lane original contraction joints and thirty-nine percent of the northbound lane original contraction joints are exhibiting medium-severity transverse joint and corner spalling; a few are displaying high-severity spalling. Corner spalling is more prevalent in the northbound lanes and is typically located on the leave side of the joint. This indicates that the relief joints did not reduce joint spalling. Original contraction joint fault measurements averaged 0.22 in [5.6 mm] in the outer lane and 0.14 in [3.6 mm] in the inner lane. More than half of the outer lane joints are unacceptably rough.

Roughness measurements were not available for this pavement, but PSIs were computed in 1983 (northbound average PSI = 3.7, southbound average PSI = 3.4). A skid number of 42 (good) was obtained in 1985.

The pressure relief joints have closed to an average width of 1.5 in [38 mm]and have faulted an average of 0.13 in [3.3 mm]. The filler in these joints is absent and the joints are full of foreign materials. Most of the joints display low-severity joint and corner spalling. The immediately adjacent contraction joints (on either side of the relief joint) have opened to an average width of 0.8 in [20 mm], and most of these adjacent joints are filled with incompressibles. Figure 47 is a plot of joint width versus sample unit stationing and is representative of the effect of the relief joints on the adjacent regular contraction joints within this project. Generally, the closer the joint to the relief joint, the wider the joint. Figure 48 provides a similar picture for the southbound sample unit with no pressure relief, but where a blowup occurred. The adjacent joints are very wide near the blowup and then become narrower as the distance from the blowup increases.

The Louisiana Department of Transportation and Development (LDOTD) has monitored the pressure relief joint width for several of the relief joints (99). This information is presented in Table 4. A continual closing of most of the joints has occurred as noted before.

The general conclusions of the study by LDOTD on the use of the pressure relief joints were (99):

1. they were effective in eliminating blowups, therefore saving the cost of blowup repair and eliminating possible hazard to the motoring public.

2. they were effective in prolonging the life of PCC pavements by reducing premature pavement distress due to contraction joint failure.

The results of the field surveys lend support to the first conclusion, but not the second.

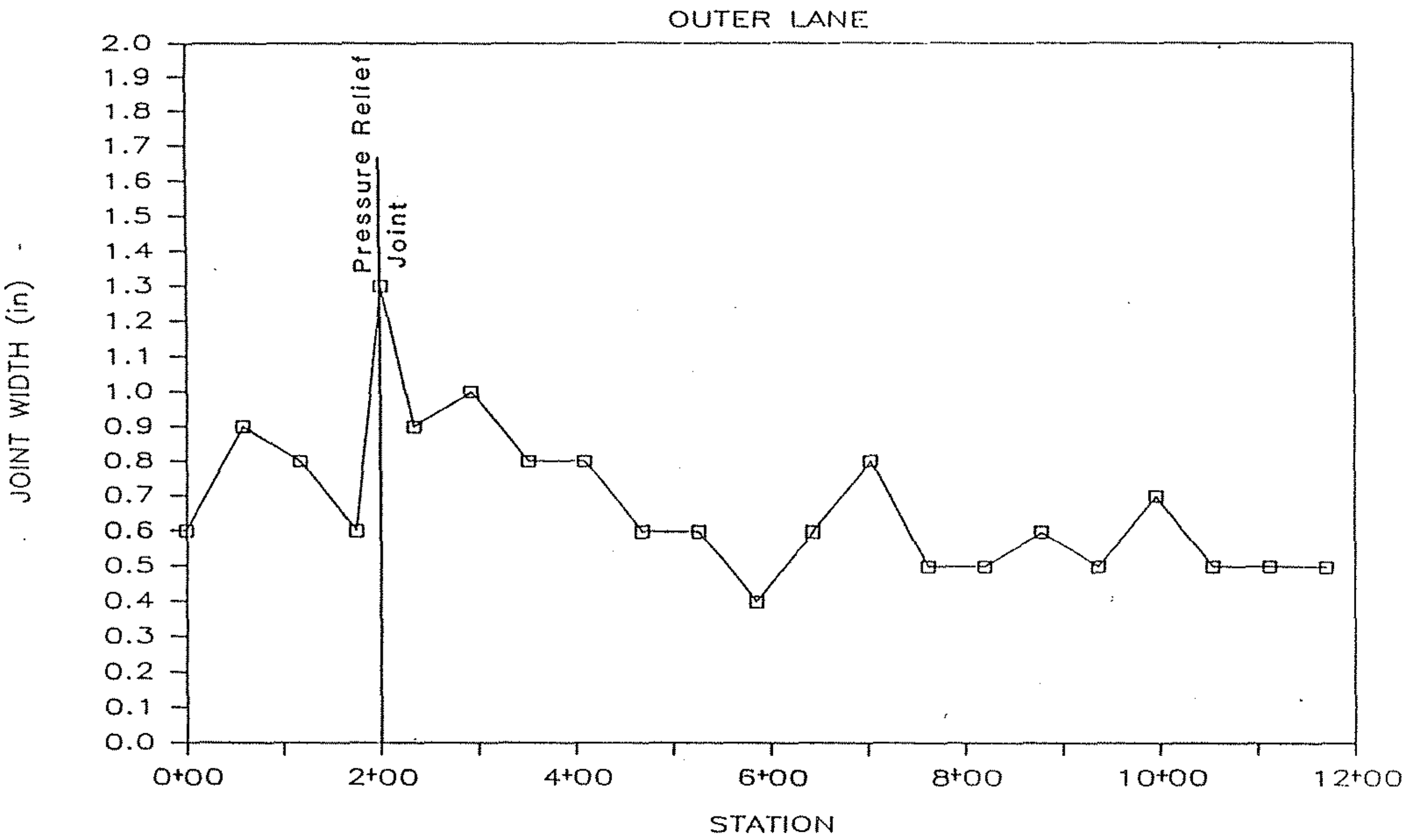


Figure 47. Plot of Joint Width Versus Station for Milepost 45.9 (Outer Lane) of LA055032 [1 in = 25.4 mm].

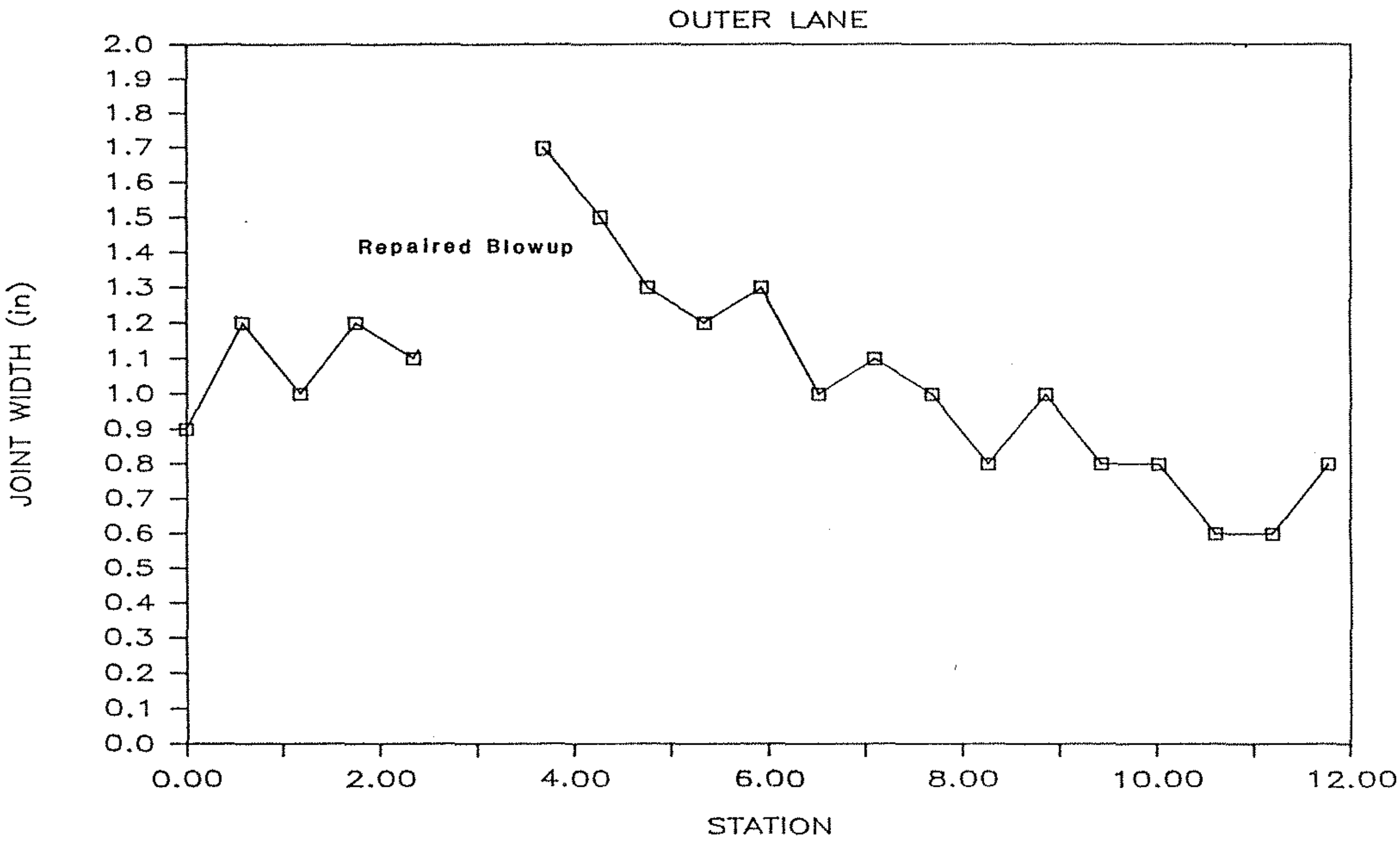


Figure 48. Plot of Joint Width Versus Station for Milepost 35.1 (Southbound Direction - Outer Lane) of LA055032 [1 in = 25.4 mm].

Table 4. Monitoring of Selected Relief Joint Widths [1 in = 25.4 mm] (99).

Relief Joint No.	Date of Measurement 10-80	8-81	9-81	1-82	7-82	1-83	9-83	9-84
2	4.0	2.9	3.0	3.1	2.7	2.8	3.0	2.6
6	4.0	3.8	3.9	3.6	3.2	3.3	2.8	2.7
7	4.0	2.3	2.3	2.4	1.3	1.5	1.2	1.25
8	4.0	2.7	2.4	3.8	1.8	2.0	1.5	1.4
9	4.0	4.4	4.1	4.1	3.7	3.6	3.1	2.6
10	4.0	0.8	0.8	0.8	0.9	0.9	0.7	0.53
15	4.0	0.8	0.9	0.9	0.9	1.2	1.0	0.66
20	4.0	0.8	0.9	1.1	0.8	0.8	0.9	0.90
25	4.0	2.0	1.9	2.0	1.3	1.7	1.2	1.25
30	4.0	2.3	2.4	2.4	1.7	2.1	1.7	1.50
35	4.0	2.2	2.4	2.4	1.9	1.9	1.6	1.60
39	4.0	1.3	1.3	1.3	0.8	1.0	0.8	1.00
43	4.0	1.3	1.4	1.1	0.9	1.2	0.7	0.80
Average	4.0	2.1	2.1	2.2	1.7	1.8	1.6	1.40

Note: Forty-five relief joints were cut in October, 1980, and these were numbered from north to south. Thus, the number of the relief joint can give an approximate location of the relief joint within the project.

The pressure relief joints installed in the northbound lanes have been effective in preventing the occurrence of blowups, but have not reduced joint deterioration or significantly improved pavement serviceability. The adjacent regular contraction joints near the relief joints have opened and, where the sealant is absent, are filled with incompressibles. However, the joints near the blowup have also opened up and are filled with incompressibles. Spalling and faulting of the transverse joints are approximately the same in both the northbound and southbound lanes. This is expected, since pressure was eventually relieved in both directions. No cracks have deteriorated appreciably in either direction. The two northbound and southbound sections have comparable PSI values.

It is doubtful that the pavement expansion problem would have developed with a good joint maintenance program in place. It is likely that cleaning the incompressibles from the joints and resealing prior to the occurrence of blowups would have significantly reduced the deterioration of this pavement and resulted in a higher overall level of serviceability.

The expected remaining life of the pressure relief joints is about 8 to 10 years (6.2 million ESALs) before they are fully closed. Blowups may again occur if new relief joints are not cut at that time. Perhaps a better solution would be to clean out the transverse joints with high pressure water to remove the existing incompressibles and then reseal the joints to keep out incompressibles.

MI127 - The original pavement was 9-in [229 mm] JRCP with 99-ft [30-m] contraction joints and was constructed and opened to traffic in 1956. Pressure relief joints were installed in the southbound lanes in 1972 using diamond saws. The northbound lanes served as a "control" section. The pavement served the Michigan Department of State Highways and Transportation as an experimental section to evaluate the merit of preventive maintenance of concrete pavement joints. Full-depth repairs (precast and cast-in-place) were placed in the southbound (relieved) lanes as required just prior to installation of the relief joints. They were also placed in the northbound (control) lanes, as required, on an annual basis. These repairs included 2 in [51 mm] undowelled expansion joints for both the approach and leave joints. Details concerning the design and performance of the full-depth repairs installed on this project are included in other portions of this report. Additional information is also available in Reference 115.

The four-inch [102-mm] wide, polyethylene-filled pressure relief joints were placed at intervals such that each was a minimum of 200 ft [61 m] and a maximum of 1200 ft [366 m] from the nearest full-depth repairs, since these repairs served to relieve compressive stresses locally. The relief joints were also placed 6 ft [1.8 m] away from the nearest contraction joint. The concrete between the saw cuts was removed using air hammers and hand tools, the joint was cleaned, and a lubricant-adhesive was applied to the joint walls. The filler was then placed in the joints and the excess filler was cut off flush with the pavement.

By 1985, the pressure relief joints had sustained about 2.8 million 18-kip [80-kN] ESALs in the outer lane and 0.4 million in the inner lane while the original pavement had received more than 5 million in the outer lane and 0.7 million in the inner lane.

The condition surveys conducted in 1985 revealed all levels of transverse slab cracking in both directions, although the density and severity of cracking found in the southbound (relieved) lanes was much higher than that found in the northbound (control) lanes, as shown below [1 ft = 0.30 m; 1 mile = 1.6 km]:

Avg. No. of Cracks/Mile
(Avg. No. of Cracks/99 ft slab)

	Southbound		Northbound	
Crack Severity	Outer	Inner	Outer	Inner
Low (hairline)	134 (2.52)	53 (0.99)	79 (1.49)	41 (0.77)
Medium (working)	57 (1.06)	75 (1.41)	18 (0.33)	18 (0.33)
High (badly spalled)	34 (0.64)	34 (0.64)	9 (0.17)	6 (0.11)
TOTAL	225 (4.22)	162 (3.04)	106 (1.99)	65 (1.21)

Figure 49 is a photo of a high-severity transverse crack found in the southbound lanes. One low-severity blowup was also found in the southbound lanes. Low-severity scaling was occasionally identified, but corner breaks, "D" cracking and reactive aggregate problems were not observed.

The contraction joint sealant was generally intact, but nevertheless, the joint often contained incompressibles. These regular contraction joints consistently exhibited medium-severity transverse joint and corner spalling throughout all of the surveyed sample units (see Figure 50). The northbound lanes, however, displayed much higher severity spalling and pressure damage. Figure 51 shows the average length of transverse joint spalling observed at each contraction joint and how it continued to increase in both the relieved and unrelieved lanes after the placement of the relief joints. The rate of increase appears to be slightly lower for the relieved lanes. As mentioned above, the spalls observed in 1985 were more severe (deeper and longer over a given width) in the northbound lanes. Comparing the location and severity of the spalls in the two directions suggests that, although the average length of spalling at the joints is nearly equal, the northbound spalls are more pressure-related than the southbound spalls.

Contraction joint faulting was generally about twice as high in the northbound (unrelieved and more damaged) lanes as in the southbound (0.11 in [2.8 mm] versus 0.05 in [1.3 mm]); however, nearly all of the joints met the acceptance criteria for smoothness. The largest faults and joint widths were often found near relief joints or full-depth repairs.

Longitudinal joint faulting was identified throughout the southbound sample units and at a few locations in the northbound samples. This faulting was generally in the range of 0.25-0.50 in [6.4-12.8 mm].

Surface friction was measured in October, 1979 using a locked wheel trailer with ASTM E274 standard tire. Skid numbers ranged from 40 to 54 (good) in both lanes of each direction. Recent roughness measurements were unavailable and Present Serviceability Index (PSI) values could not be calculated from the data provided.

The pressure relief joints installed in 1972 have closed from 4 in [102 mm] to an average width of 0.55 in [14 mm], and have faulted an

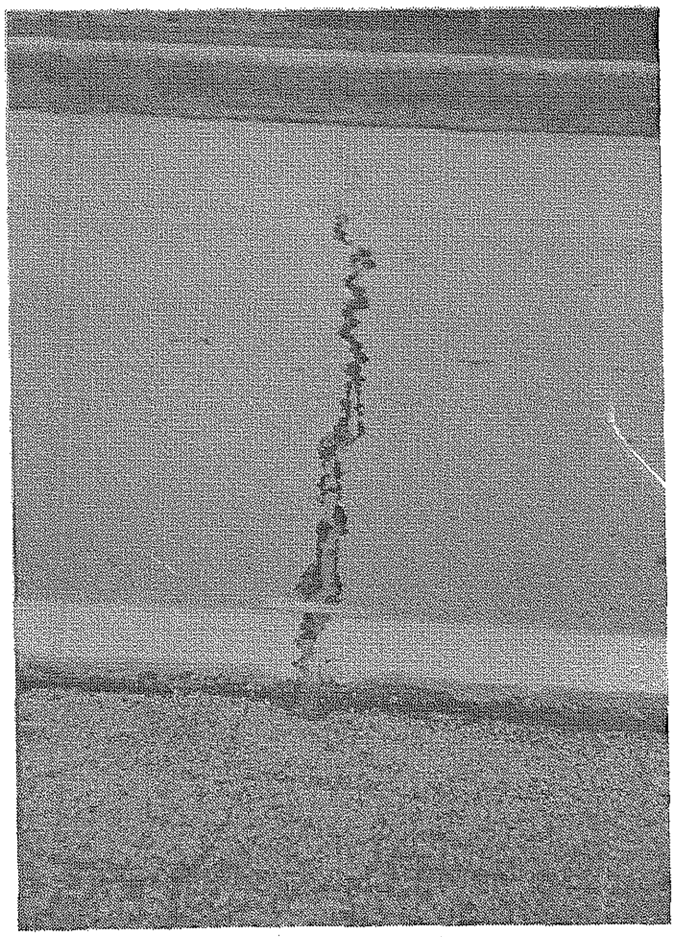


Figure 49. Photo of High-Severity Crack in Sample Unit 3 (Station 9+60) on MI127.

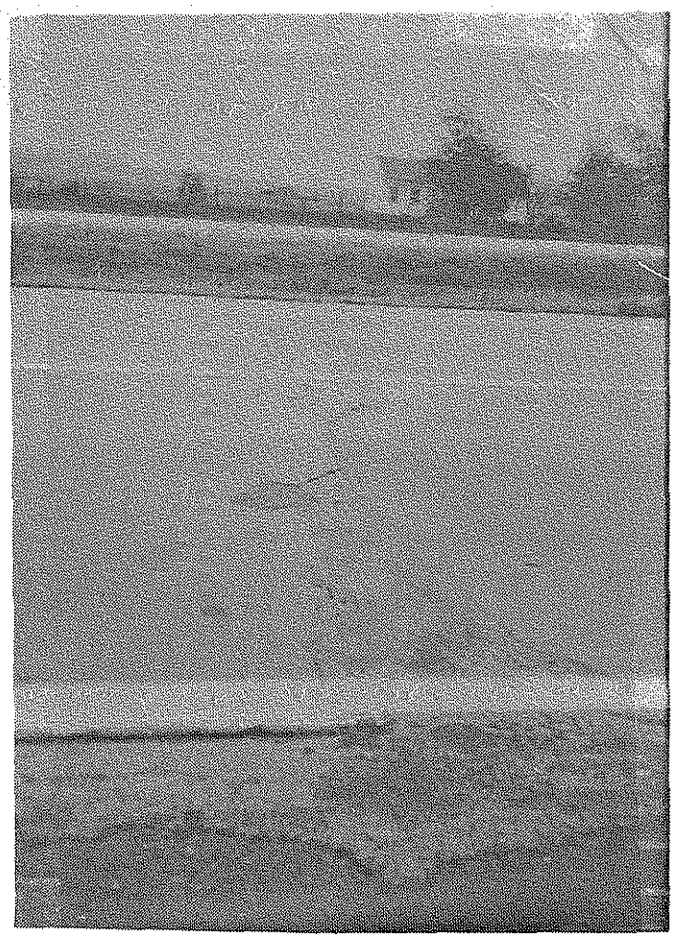


Figure 50. Photo of Regular Contraction Joint in Sample Unit 5 (Station 4+06) on MI127.

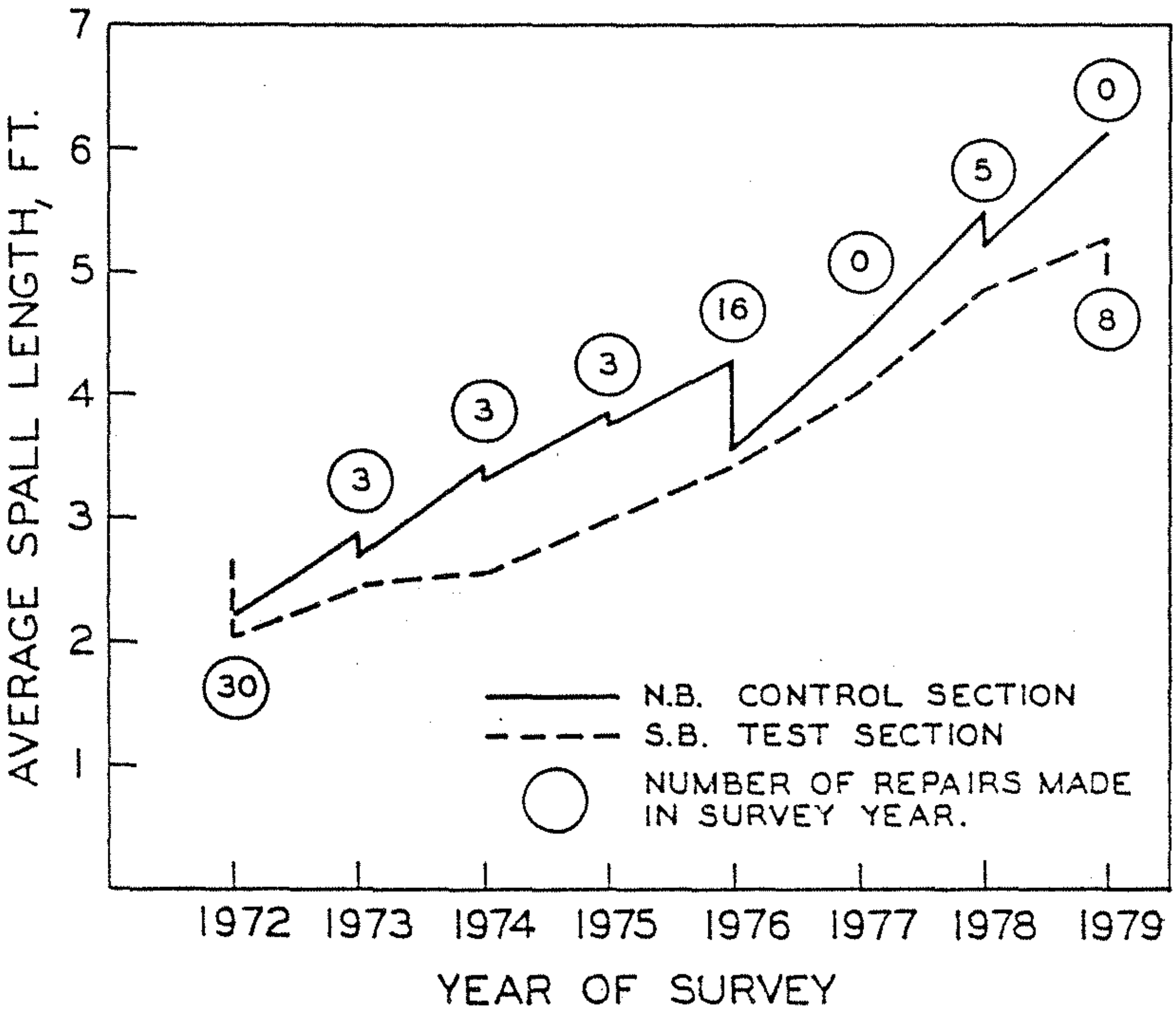


Figure 51. Average Joint Spall Length Before and After Repair for 1972 - 1979 (115) [1 ft = 0.3 m].

average of 0.27 in [7 mm]. The joint filler was still intact and was keeping incompressibles from entering. Transverse joint and corner spalling were not exhibited in conjunction with any of the relief joints.

Summer and winter measurements of the relief joints were made each year between 1972 and 1979 and these measurements are summarized in Figure 52 (115). This figure shows that the largest amount of relief joint closure generally took place in the first year after installation. It also indicates that the period of effectiveness of these relief joints ranged from about 3 to 7 years, as indicated by the constant joint openings observed after this time period.

Reference 115 indicates that the width of contraction joints located near relief joints (and where the slab was free of transverse cracks with fractured steel) was directly related to the distance of the contraction joint from the relief joint. The average width of the nearest joint was found to be 1.14 in [29.0 mm], the second joint width averaged 1.04 in [26.4 mm] and the third joint width averaged 0.88 in [22.4 mm]. All of these measurements were taken at a temperature of about 45°F [7°C]. No such pattern was observed in 1985 but it is likely that the joints were tighter because the survey was conducted on a warmer day and that the widths have stabilized because the surrounding slabs all contain deteriorated transverse cracks.

The pressure relief joints installed in 1972 were an effective means of preventing blowups and pressure-related joint damage from occurring in the southbound lanes. Fifty percent of the joints in the southbound lanes were deteriorated, compared with 77% in the northbound lanes. Data collected by MDOT indicates that these joints served their intended function for 5 to 7 years before they closed permanently. In 1979, eight full-depth repairs were placed at joints in the southbound lanes to address spalling problems. The placement of additional relief joints at this time may have slowed the rate of joint spalling.

While the relief joints have greatly reduced the deterioration of the transverse joints, they have allowed the transverse cracks to deteriorate to the point where the rehabilitated southbound lanes will now require more repair than the unrelieved northbound lanes, which exhibit badly spalled joints but little slab cracking. It is possible that a narrower relief joint design would have protected the transverse joints and prevented the transverse cracks from deteriorating as much.

Relief joints were not placed at constant intervals on this project, but at intervals that would best benefit unrepaired portions of the pavement (i.e., they were not placed near full-depth repairs where the installation of such repairs would have alleviated any compressive stresses in the immediate vicinity). This is evidenced in the sample unit taken near station 759+00 in the southbound lanes, where the relief joint is located 300 ft [91 m] from the nearest repair. This distance could probably have been increased to 500 ft [152 m] to reduce the possibility of over-relieving the pavement in this area.

The pressure relief joints exhibited high levels of faulting (average relief joint faulting = 0.27 in [6.9 mm]) which might have been reduced with the inclusion of load transfer devices.

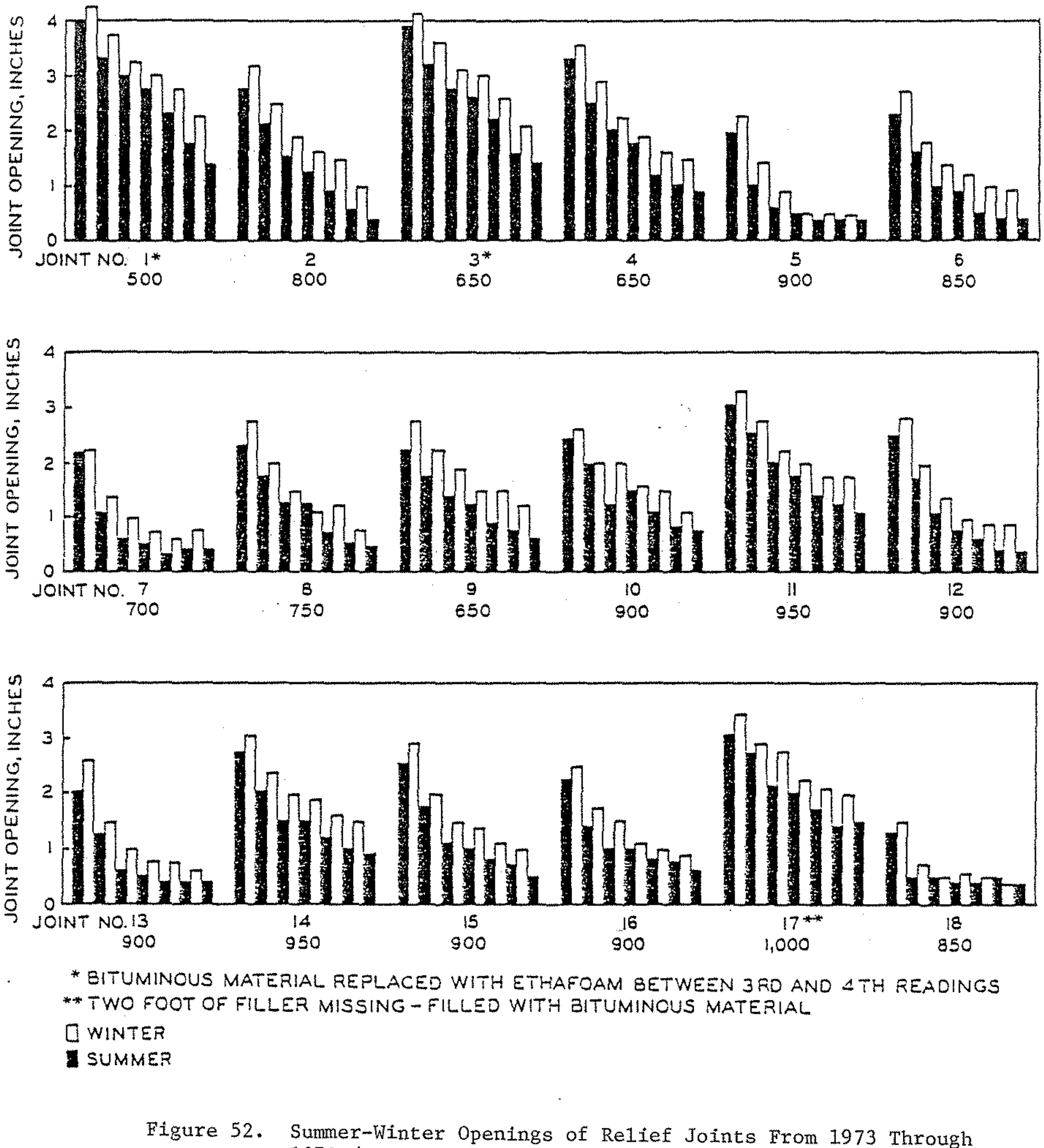


Figure 52. Summer-Winter Openings of Relief Joints From 1973 Through 1979 (Numbers Below Each Joint Indicate Pavement Length Contributing to the Joint Closure) (115) [1 in = 25.4 mm].

This pavement was 16 years old when the relief joints were installed. This project presents evidence that the installation of relief joints should be kept to a minimum when well developed transverse cracks are present. An alternative to the installation of pressure relief joints in this case would have been to thoroughly clean the transverse joints and working cracks to remove the materials that are causing the pavement to build pressure, and then reseal them. The placement of full-depth repairs with load transfer devices and without expansion joints would have relieved any built-up pressure without allowing the adjacent cracks to open further.

The overuse of relief and repair expansion joints has caused excessive breakdown of the transverse cracks to the point where the rehabilitated lanes now require more additional rehabilitation than the non-rehabilitated lanes. In fact, successful rehabilitation of the northbound lanes could be accomplished with only full-depth joint repairs, installation of subdrains and diamond grinding while the southbound lanes require extensive slab and joint repairs, a structural overlay or reconstruction.

Suggested improvements to the repair and relief joint designs include:

1. Provision of load transfer across all joints.

2. Narrower relief and expansion joint widths to prevent excessive movement of adjacent joints and cracks.

3. Additional separation between relief joints and concurrently placed full-depth repairs.

NE080189 - The original pavement was 9-in [230 mm] JRCP with 46.5-ft [14.2 m] contraction joints and was constructed and opened to traffic in 1965. Four-and-a-half-inch [114-mm] wide pressure relief joints were installed near mid-slab in 1981 at 2000-ft [610-m] intervals using a wheel saw to reduce expansive pressures caused reactive aggregate. A preformed cellular joint filler (Flex Lok) was installed. Full-depth repairs without mechanical load transfer devices were placed at about 27% of the driving lane joints and 8% of the passing lane joints to address reactive aggregate spalls. Additional repairs were placed in 1985 to address further joint deterioration. The joints were also resealed in 1985 (after the survey was completed) using a hot-poured sealant (Roadsaver 213, manufactured by Crafco, Inc.) conforming to ASTM D3405-78.

By 1985, the pressure relief joints had sustained about 4 million 18-kip [80-kN] ESALs in the outer lane and 1 million in the inner lane. The original pavement had received more than 13 million in the outer lane and 2.5 million in the inner lane.

The condition surveys conducted in 1985 showed reactive aggregate distress (see Figure 53), low- and some medium-severity transverse cracking, transverse joints with extruded sealant and incompressibles in some joints, medium-severity transverse joint and corner spalling (mostly in the outer lane) and localized areas of scaling. The remaining transverse contraction joints were faulted an average of 0.02 in [0.5 mm] and were open an average of 0.3 in [7 mm]. An average Roughness Index of 120 (good) was obtained with a Mays Ride Meter in 1985. A skid number of 44 (good) was obtained with a locked wheel skid trailer in 1984.

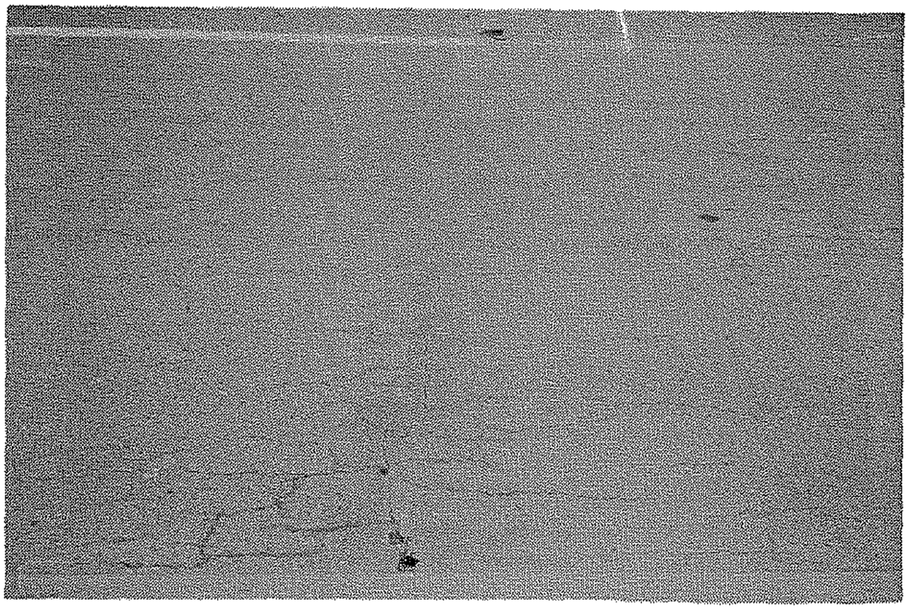


Figure 53. Photo of Reactive Aggregate Distress Observed on NE080189.

The pressure relief joints that were installed in the 1981 have closed to an average width of 0.8 in [20 mm] and have faulted an average of 0.09 in [2.3 mm], more than four times the contraction joint average faulting. The filler is still intact and only low severity spalling was observed. The adjacent original contraction joints have not opened appreciably wider than project average joint width. Table 5 and Figure 54 summarize and illustrate the closure of the pressure relief joints on this project over time, as measured by the Nebraska Department of Roads.

The pressure relief joints were an appropriate measure taken to combat the compressive stresses which result from the pavement growth caused by the reactive aggregate. This need for relief was evidenced by the amount of closure of the relief joints (average amount of closure 3.7 in [94 mm]), although it is interesting to note that the average amount of closure is approximately equal to the total amount that the intermediate contraction joints have opened (0.1 in [2.5 mm] opening x 45 joints = 4.5 in [124 mm]).

Since full-depth repairs were to be placed that same year (1981), it is possible that the installation of the repairs would have relieved any built-up pressure. The relief joints do not appear to have had an adverse affect on the performance of the rest of the pavement.

Overall, this rehabilitation project is considered successful due to the good performance of the full-depth repairs. Most of the joints in both lanes are smooth, and spalling is not of concern. The pressure relief joints were probably an appropriate effort to reduce pressure damage, although they have faulted rapidly and the adjacent contraction joints have opened slightly.

This pavement can be expected to provide good serviceability for 2 to five years from the date of survey before major rehabilitation is required to address joint and crack spalling caused by the presence of the reactive aggregate.

NE080210 - The original pavement was 9-in [230] JRCP with 46.5-ft [14.2 m] contraction joints and was constructed and opened to traffic in 1964. Four-and-a-half inch [114-mm] wide pressure relief joints were installed near mid-slab in 1980 at 2000-ft [610-m] intervals using a wheel saw to reduce the expansive pressures caused by reactive aggregate. A preformed cellular joint filler (Flex Lok) was installed. Full-depth repairs without load transfer devices were placed in 1979 at nearly 90% of the joints in either lane to address spalls caused by reactive aggregate problems. Some major cracks were also repaired. The design and performance of the full-depth repairs is described in detail elsewhere in this report.

By 1985, the pressure relief joints had sustained about 6 million 18-kip [80-kN] ESALs in the outer lane and 0.5 million in the inner lane. The original pavement had received more than 15 million in the outer lane and 1 million in the inner lane.

The condition surveys conducted in 1985 found low-severity transverse cracks present in about half of the surveyed slabs and advanced reactive aggregate distress throughout the pavement, although it was most severe near the joints.

Table 5. Monitoring of Closure of All Pressure Relief Joints on NE080189 [1 in = 25.4 mm].

	EASTBOUND			WESTBOUND		
DATE	AVG CLOSURE	TOTAL CLOSURE	WIDTH OF CUT	AVG CLOSURE	TOTAL CLOSURE	WIDTH OF CUT
APR `81	0.50	0.50	3.63	0.49	0.49	3.67
JUN `81	0.72	1.22	2.91	1.01	1.50	2.75
JUL `81	0.55	1.77	2.36	0.54	2.04	2.21
AUG `81	0.23	2.00	2.13	0.29	2.33	1.92
SEP `81	0.09	2.09	2.04			
NOV `81	+0.11	1.98	2.15	+0.11	2.22	2.03
DEC `81	+0.13	1.85	2.28	+0.11	2.11	2.14
FEB `82	0.03	1.88	2.25	0.11	2.22	2.03
APR `82	0.13	2.01	2.12	0.15	2.37	1.88
MAY `82	0.35	2.36	1.77	0.42	2.79	1.46
JUN `82	0.23	2.59	1.54	0.21	3.00	1.25
JUL `82	0.37	2.96	1.17	0.57	3.57	0.68
SEP `82	0.12	3.08	1.05	+0.12	3.45	0.80
OCT `82	+0.09	2.99	1.14	+0.13	3.32	0.93
NOV `82	+0.13	2.86	1.27	+0.12	3.20	1.05
JAN `83	+0.12	2.74	1.39	+0.16	3.04	1.21
MAR `83	0.10	2.84	1.29	0.12	3.16	1.09
JUN `83	0.53	3.37	0.76	0.51	3.67	0.58
JUL `83	0.30	3.67	0.46	0.23	3.90	0.35
FEB `84	+0.19	3.48	0.65	+0.16	3.74	0.51
MAY `84	0.17	3.65	0.48	0.18	3.92	0.33
DEC `84	0.03	3.68	0.45	+0.14	3.78	0.47

Note: A positive (+) value indicates opening of the relief joint.

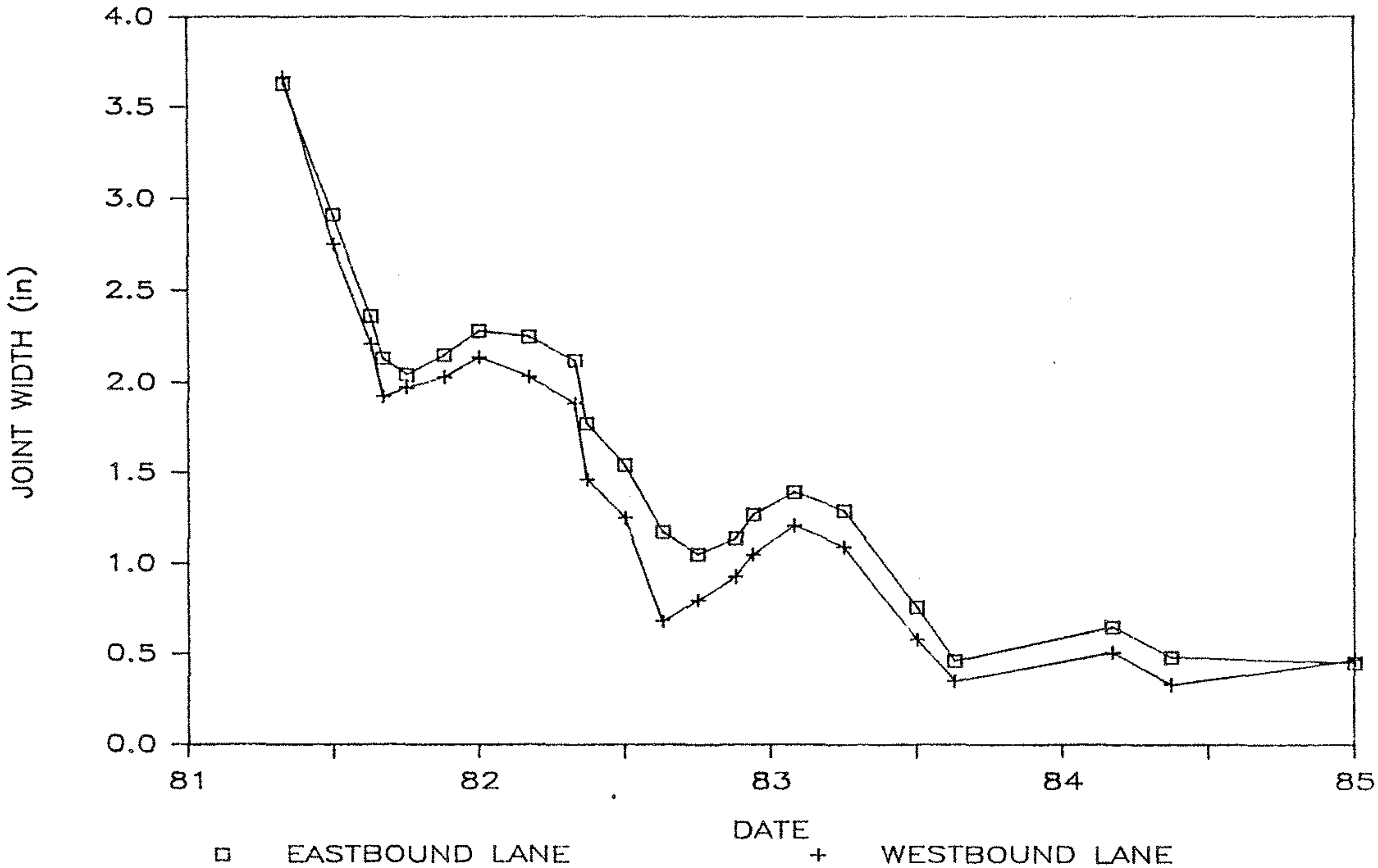


Figure 54. Pressure Relief Joint Closure Over Time on NE080189 [1 in = 25.4 mm].

The contraction joint sealant was present in all of the original contraction joints, keeping incompressibles from infiltrating. Medium-severity transverse joint spalling (due to reactive aggregate and pressure damage) was found at many of the remaining original contraction joints. The average faulting and average joint width measurements for these original contraction joints in the outer lane was 0.03 in [0.76 mm] and 0.22 in [5.5 mm], respectively for both lanes.

Despite the low amount of rainfall and AASHTO classification A-1-a subgrade found at the site, low-severity pumping was observed throughout the project at the regular contraction joints and near some of the full-depth repairs.

An average Roughness Index of 163 (good) was measured in April, 1985 using a Mays Ride Meter. A skid number of 46 (good) was measured in June, 1984 using a locked-wheel trailer with ASTM E274 standard tire. PSI values were computed to be 3.8 from the roughness data.

The pressure relief joints that were installed in 1980 have closed from 4.5 in [114 mm] width to an average width of 0.9 in [23 mm] and have faulted an average of 0.17 in [4.3 mm]. Table 6 presents a summary of the closure of the pressure relief joints for the entire project, as measured by the Nebraska Department of Roads. Figure 55 illustrates the relief joint closure over time. The filler is still intact in most of the relief joints and is keeping incompressibles from infiltrating. Spalling was typically of low severity at the relief joints, but in a few cases medium-severity spalling was present.

The contraction joints adjacent to the relief joints have not opened appreciably greater than the average width 0.23 in [5.8 mm]. This indicates that the effective slab length in compression is greater than or equal to the relief joint spacing.

The pressure relief joints were an appropriate measure taken on this project to combat the expansion caused by the reactive aggregate and evidenced by the amount of closure of the relief joints (average amount of closure 4.0 in [102 mm]) without significant opening or faulting of adjacent contraction joints. However, the full-depth repairs placed in 1979 are believed to have relieved any built-up pressure.

Overall, this rehabilitation project is considered successful. The full-depth repairs and pressure relief joints are performing well and the transverse joints are generally smooth. However, some spalling is developing at the approach and leave slabs outside of the repairs as the reactive aggregate continues to cause deterioration. Additional full-depth repairs and/or overlay will be necessary within the next few years.

Less closure and lower rates of closure were found in the relief joints on this project (as determined by the Nebraska Department of Roads) than the preceding one (NE080189). This can probably be attributed to the placement of full-depth repairs before the installation of the relief joints on this project. The full-depth repairs allowed for some of the pressure to be relieved, but the relief joints were still necessary to allow for the expansion of the pavement caused by reactive aggregate problems.

Table 6. Monitoring of Closure of All Relief Joints on NE080210 [1 in = 25.4 mm].

	EASTBOUND			WESTBOUND		
DATE	AVG CLOSURE	TOTAL CLOSURE	WIDTH OF CUT	AVG CLOSURE	TOTAL CLOSURE	WIDTH OF CUT
MAR `80	0.28	0.28	4.69	0.16	0.16	4.35
APR `80	0.18	0.46	4.51	0.10	0.26	4.25
MAY `80	0.45	1.07	3.90	0.72	1.53	2.98
JUN `80	0.46	1.53	3.44	0.67	2.20	2.31
JUL `80	0.38	1.91	3.06	0.40	2.60	1.91
AUG `80	0.14	2.05	2.92	0.12	2.72	1.79
SEP `80	+0.13	1.92	3.05	+0.11	2.61	1.90
JAN `81				+0.11	2.50	2.01
APR `81	0.30	2.22	2.75	0.30	2.80	1.71
JUN `81	0.35	2.57	2.40	0.41	3.21	1.30
JUL `81	0.23	2.80	2.17	0.16	3.37	1.14
AUG `81	0.16	2.96	2.01	0.15	3.52	0.99
SEP `81	0.13	3.09	1.88	0.13	3.65	0.86
NOV `81	+0.11	2.98	1.99	+0.12	3.53	0.98
DEC `81				+0.10	3.43	1.08
MAY `82	0.23	3.21	1.76	0.16	3.59	0.92
JUN `82	0.14	3.35	1.62	0.15	3.74	0.77
JUL `82	0.24	3.59	1.38	0.29	4.03	0.48
SEP `82	+0.15	3.44	1.53	+0.14	3.89	0.62
OCT `82				+0.13	3.76	0.75
NOV `82				+0.10	3.66	0.85
JAN `83	+0.15	3.29	1.68	+0.15	3.51	1.00
MAR `83	0.15	3.44	1.53	0.10	3.61	0.90
JUN `83	0.27	3.71	1.26	0.26	3.87	0.64
JUL `83	0.18	3.89	1.08	0.16	4.03	0.48
SEP `83	0.13	4.02	0.95	0.11	4.14	0.37
DEC `83				+0.18	3.96	0.55
JAN `84	+0.17	3.85	1.12			
MAY `84	0.15	4.00	0.97	0.14	4.10	0.41
DEC `84	+0.01	3.99	0.98	+0.11	3.99	0.52

Note: A positive (+) value indicates opening of the relief joints.

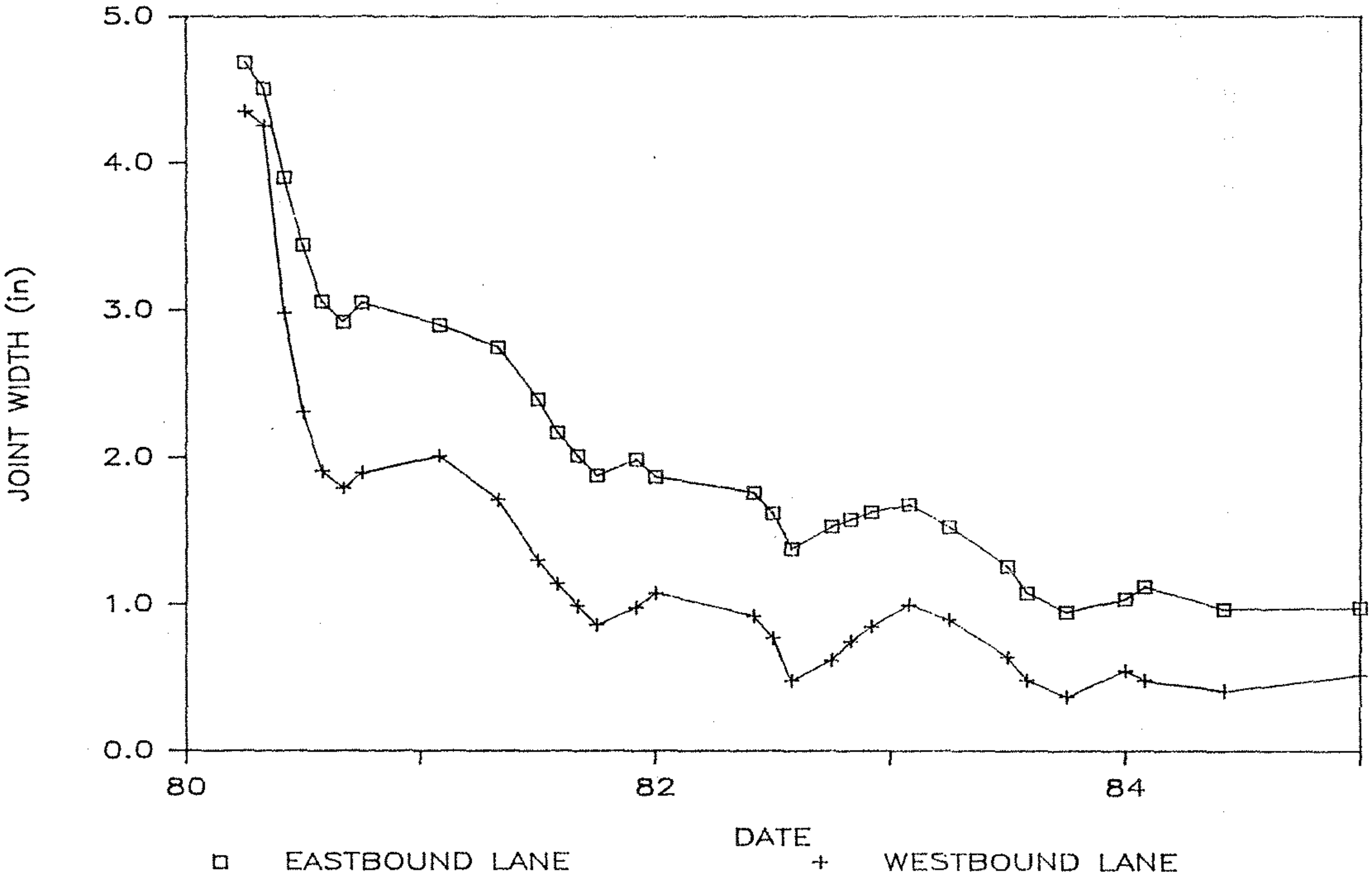


Figure 55. Pressure Relief Joint Closure Over Time on NE080210
[1 in = 25.4 mm].

NE080256 -The original pavement was 9-in [230 mm] JRCP with 46.5-ft [14.2 m] contraction joints and was constructed and opened to traffic in 1963. Four-and-a-half-inch [114-mm] wide pressure relief joints were installed near mid-slab in 1980 at 2000-ft [610-m] intervals using a wheel saw to reduce the expansive pressures caused by reactive aggregate. A preformed cellular joint filler (Flex Lok) was used. Full-depth repairs without mechanical load transfer devices were placed in 1980 at roughly 36% of the outer lane joints and 15% of the inner lane joints to address spalls caused by reactive aggregate problems. Some major cracks were also repaired.

By 1985, the pressure relief joints had sustained about 6 million 18-kip [80-kN] ESALs in the outer lane and 0.5 million in the inner lane. The original pavement had received about 17 million ESALs in the outer lane and nearly 1.5 million ESALs in the inner lane.

The condition surveys conducted in 1985 found low-severity transverse cracks present in about half of the surveyed slabs and advanced reactive aggregate distress throughout the pavement, although it was most severe near the joints.

The contraction joint sealant was present in most of the original contraction joints, keeping incompressibles from infiltrating. However, medium-severity transverse joint spalling (due to reactive aggregate and pressure damage) was found at many of the remaining original contraction joints. The average faulting and average joint width measurements for the outer lane original contraction joints was 0.04 in [1.0 mm] and 0.24 in [6.0 mm], respectively, while the average faulting and average joint width measurements for the inner lane original contraction joints was 0.04 in [1.0 mm] and 0.28 in [7.1 mm], respectively.

An average Roughness Index of 176 (fair) was measured in April, 1985 using a Mays Ride Meter. A skid number of 41 (good) was measured in June, 1983 using a locked-wheel trailer with ASTM E274 standard tire.

The pressure relief joints that were installed in 1980 have closed from 4.5 in [114 mm] width to an average width of 0.86 in [22 mm] and have faulted an average of 0.18 in [4.6 mm]. Table 7 presents a summary of the closure of the pressure relief joints for the entire project, as measured by the Nebraska Department of Roads. Figure 56 illustrates the relief joint closure over time. The filler is still intact in most of the relief joints and is keeping incompressibles from infiltrating. Spalling was generally of medium severity.

The contraction joints adjacent to the relief joints have not opened appreciably greater than the average width 0.22 in [6.0 mm]. This indicates that the effective slab length in compression is greater than or equal to the relief joint spacing.

The pressure relief joints placed in 1980 were an appropriate measure taken on this project to combat the expansion caused by the reactive aggregate, although the placement of the full-depth repairs in 1980 probably aided in relieving any built-up pressure.

Table 7. Monitoring of Closure of All Relief Joints on NEO80256 [1 in = 25.4 mm].

	EASTBOUND			WESTBOUND		
DATE	AVG CLOSURE	TOTAL CLOSURE	WIDTH OF CUT	AVG CLOSURE	TOTAL CLOSURE	WIDTH OF CUT
OCT `80				0.19	0.19	4.66
NOV `80	0.06	0.06	4.73	0.16	0.35	4.50
FEB `81				0.19	0.54	4.31
MAR `81	0.24	0.30	4.49	0.37	0.91	3.94
APR `81	0.71	1.01	3.78	1.07	1.98	2.87
JUN `81	0.55	1.56	3.23	0.51	2.49	2.36
JUL `81	0.43	1.99	2.80	0.64	3.13	1.72
AUG `81	0.15	2.14	2.65	0.16	3.29	1.56
SEP `81	0.13	2.27	2.52	0.12	3.41	1.44
NOV `81	+0.15	2.12	2.67	+0.12	3.29	1.56
DEC `81				0.08	3.21	1.64
FEB `82				0.10	3.31	1.54
APR `82				+0.10	3.21	1.64
MAY `82	0.37	2.49	2.30	0.32	3.53	1.32
JUN `82	0.30	2.79	2.00	0.27	3.80	1.05
JUL `82	0.28	3.07	1.72			
SEP `82				+0.08	3.72	1.13
OCT `82				+0.11	3.61	1.24
DEC `82	0.12	2.95	1.84	+0.14	3.47	1.38
JAN `83	+0.13	2.82	1.97	+0.12	3.35	1.50
MAR `83	0.11	2.93	1.86	0.06	3.41	1.44
JUN `83	0.29	3.22	1.57	0.27	3.68	1.17
JUL `83	0.23	3.45	1.34	0.11	3.79	1.06
AUG `83	0.11	3.56	1.23	0.09	3.88	0.97
JAN `84	+0.22	3.34	1.45	+0.11	3.77	1.08
MAY `84	0.19	3.48	1.31	0.15	3.92	0.93
DEC `84	0.06	3.54	1.25	+0.12	3.80	1.05

Note: A positive (+) value indicates opening of the relief joints.

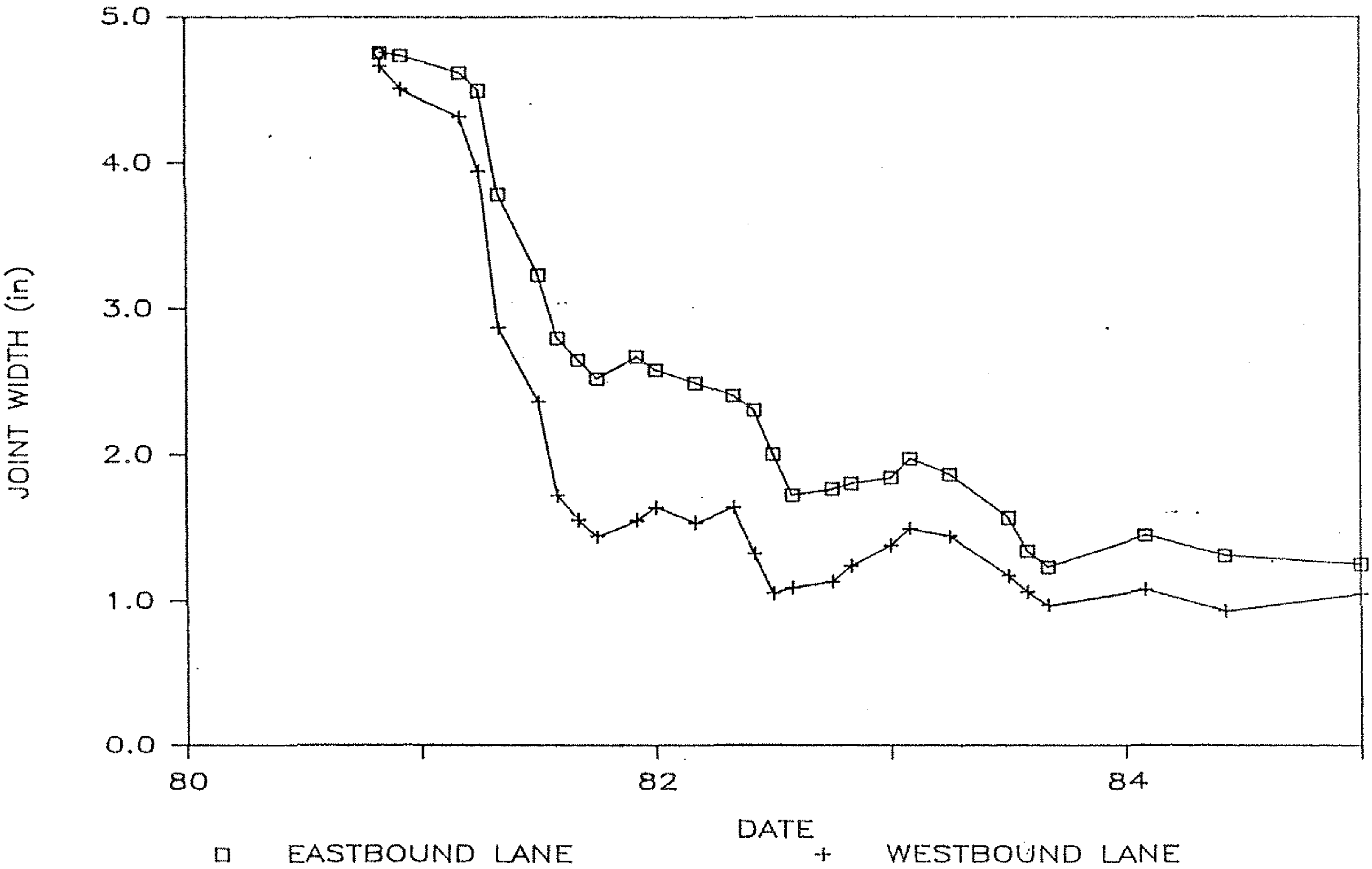


Figure 56. Pressure Relief Joint Closure Over Time on NE080256 [1 in = 25.4 mm].

Overall, this rehabilitation project is considered successful due to the good performance of the full-depth repairs and absence of much recent pressure damage. Most of the joints in both lanes are smooth, and spalling is only a problem on the approach and leave slabs outside of the repairs. The expected life of this pavement would be approximately six to nine years from the installation of the full-depth repairs (8.4 to 12.6 million ESALs). Additional full-depth repairs and/or an overlay will be necessary as early as 1986 or as late as 1989.

The performance of the pressure relief joints has been satisfactory for this project and the two preceding it (NE080189 and NE080210). It is observed that the rate of closure and the amount of closure is less for this project (NE080256) where full-depth repairs and pressure relief joints were installed the same year. However, the relief joints were apparently still necessary to allow for the expansion of the pavement caused by reactive aggregate.

NE080279 - The original pavement was 9-in [230 mm] JRCP with 46.5-ft [14.2 m] contraction joints and was constructed and opened to traffic in 1964. Four-and-a-half-inch [114-mm] wide pressure relief joints were installed near mid-slab in 1980 at 2000-ft [610-m] intervals using a wheel saw to reduce the expansive pressures caused by reactive aggregate problems. A preformed cellular plastic joint filler conforming to ASTM D3204-80 was used. Full-depth repairs without mechanical load transfer devices were also placed at about 10% of the outer lane joints and 4% of the inner lane joints to address spalls caused by reactive aggregate problems. Some major cracks were also repaired. A few partial-depth repairs made of an epoxy cement were placed to address localized spalls at joints and major cracks. Finally, the transverse joints were resealed with a hot-poured sealant conforming to ASTM D3405-78. The performance of these other rehabilitation techniques is described elsewhere in this report.

By 1985, the pressure relief joints had sustained about 4 million 18-kip [80-kN] ESALs in the outer lane while the original pavement had received about 14 million in the outer lane.

The condition surveys conducted in 1985 found low- and medium-severity transverse cracks at approximately 10-20 ft [3-6 m] intervals, although some panels showed much higher levels of deterioration and are in need of immediate repair. Reactive aggregate problems (joint and corner spalling and scaling) were also found throughout the pavement, although severe spalling was not often observed.

The contraction joint sealant was present in most of the original contraction joints. As a result, transverse joint and corner spalling were generally of low severity throughout the surveyed sample units and none of the surveyed original contraction joints had been repaired, although some panel cracks had been repaired. The average faulting and joint width measurements for the original contraction joints in the outer lane were 0.05 in [0.76 mm] and 0.4 in [10.2 mm], respectively, compared to 0.01 [0.29 mm] and 0.4 in [10.2 mm] for the inner lane.

An average Roughness Index of 202 (fair) was obtained in 1985 using a Mays Ride Meter. An average skid number of 39 (marginal-good) was obtained in 1983 with a locked-wheel trailer.

The pressure relief joints installed in 1982 have closed from 4.5 in [114 mm] width to an average width of 0.65 in [16.5 mm] and have faulted an average of 0.10 in [2.5 mm]. Relief joints spaced very closely appear to exhibit much more faulting and less closure than those spaced at more typical distances (see Tables 8 and 9). The filler and sealant are missing from many of the relief joints and incompressibles have filled the joints. Low- and occasional medium-severity spalling was found at the relief joints. The adjacent contraction joints have not opened appreciably greater than the average observed width of 0.4 in [10.2 mm].

The pressure relief joints that were installed in 1982 may not have been necessary on this project because the alkali-aggregate reaction, which has been so destructive to other projects in Nebraska (i.e., NE080189, NE080210, and NE080256), was not a major problem on this project. It is known that crushed limestone (rather than North Platte River aggregate) was used in the original construction of this project, which could have been more susceptible to "D" cracking than reactive aggregate distress. Joint spalling was attributed largely to improper dowel alignment and reactive aggregate spalling and built-up pressure would have been at least partially relieved during the placement of the full-depth repairs. Finally, while the average amount of relief joint closure since placement is 3.85 in [98 mm], current joint measurements indicate that most of the contraction joints are nearly twice as wide now as when originally constructed.

While the use of relief joints on this project may be questionable, their presence has apparently not adversely affected the project. The faulting of the relief joints might have been reduced slightly if a mechanical load transfer system (i. e., dowels) had been provided.

Overall, this rehabilitation project is considered successful, due to the good performance of the full-depth repairs and joint resealing. Most of the joints in both lanes are smooth and are well-sealed. The repairs can be expected to provide another two to five years of good performance before adjacent slab spalling (due to the some type of materials durability problem) becomes unacceptable. However, some areas of the original pavement are badly cracked and require rehabilitation now.

NE080382 - The original pavement was 9-in [230 mm] JRCP with 46.5-ft [14.2 m] contraction joints and was constructed and opened to traffic in 1962. Four-and-a-half-inch [114-mm] wide pressure relief joints were installed near mid-slab in 1982 at one-mile [1610-m] intervals using a wheel saw to reduce the expansive pressures caused by reactive aggregate problems. A preformed cellular plastic joint filler conforming to ASTM D3204-80 (Flex Loc, manufactured by A.C. Horn, Inc.) was used. Several other rehabilitation techniques were also applied at the same time. Full-depth repairs without mechanical load transfer devices were placed at about 13% of the outer lane joints and 9% of the inner lane joints to address spalls caused by reactive aggregate problems. Some major cracks were also repaired. A few partial-depth repairs utilizing an epoxy cement were also placed to address localized spalls at joints and major cracks. Finally, the transverse joints were resealed with a hot-poured sealant conforming to ASTM D3405-78 (trade name "Sof-Seal," manufactured by W. R. Meadows). The design and performance of these other rehabilitation techniques is described in detail in other portions of this report.

Table 8. Summary of Joint Faults and Widths At Milepost 284.0 of NE080279 [1 in = 25.4 mm].

OUTER				INNER			
STATION	TYPE OF JOINT	FAULT (in.)	WIDTH (in.)	STATION	TYPE OF JOINT	FAULT (in.)	WIDTH (in.)
0+00	Reg Cont	0.00	0.4	0+00	Reg Cont	-0.01	0.4
0+20	Repair Ap	0.07	0.4				
0+24	Repair Lv	0.16	0.4				
0+46	Reg Cont	0.01	0.4	0+46	Reg Cont	-0.02	0.4
0+63	Pr Relief	0.33	0.6	0+61	Pr Relief	0.21	1.0
0+93	Reg Cont	-0.02	0.3	0+93	Reg Cont	0.00	0.3
1+39	Pr Relief	0.36	1.0	1+39	Pr Relief	0.08	0.7
1+86	Reg Cont	0.01	0.4	1+86	Reg Cont	-0.01	0.4
2+32	Reg Cont	0.12	0.5	2+32	Reg Cont	0.00	0.4
2+79	Reg Cont	0.00	0.4	2+79	Reg Cont	-0.01	0.4
3+25	Reg Cont	0.05	0.4	3+25	Reg Cont	-0.02	0.4
3+35	Repair Ap	0.00	0.4				
3+40	Repair Lv	0.01	0.4				
3+72	Reg Cont	0.04	0.3	3+72	Reg Cont	0.04	0.4
4+18	Reg Cont	0.02	0.3	4+18	Reg Cont	0.03	0.3
4+65	Reg Cont	0.01	0.3	4+65	Reg Cont	-0.01	0.3
4+95	Repair Ap	0.04	0.4				
4+99	Repair Lv	0.02	0.4				
5+11	Reg Cont	0.10	0.3	5+11	Reg Cont	0.00	0.3
5+58	Reg Cont	0.03	0.4	5+58	Reg Cont	0.02	0.4
5+81	Pr Relief	0.03	0.3	5+81	Pr Relief	0.01	0.4
6+04	Reg Cont	0.00	0.3	6+04	Reg Cont	0.01	0.3
6+24	Repair Ap	-0.05	0.4				
6+28	Repair Lv	0.02	0.4				
6+51	Reg Cont	0.06	0.4	6+51	Reg Cont	0.00	0.4
6+97	Reg Cont	0.08	0.3	6+97	Reg Cont	0.01	0.3
7+44	Reg Cont	0.09	0.4	7+44	Reg Cont	-0.02	0.4
7+90	Reg Cont	0.01	0.4	7+90	Reg Cont	0.02	0.3
8+37	Reg Cont	0.09	0.4	8+37	Reg Cont	0.00	0.4
8+83	Reg Cont	0.01	0.3	8+83	Reg Cont	0.03	0.3
9+30	Reg Cont	0.09	0.3	9+30	Reg Cont	0.00	0.4
9+76	Reg Cont	-0.02	0.3	9+76	Reg Cont	0.01	0.3
10+06	Repair Ap	0.06	0.4	10+06	Repair Ap	-0.05	0.3
10+10	Repair Lv	0.17	0.5	10+10	Repair Lv	0.16	0.6
10+23	Reg Cont	-0.02	0.3	10+23	Reg Cont	0.02	0.4
10+69	Reg Cont	0.00	0.3	10+69	Reg Cont	0.00	0.3

Table 9. Summary of Joint Faults and Widths At Milepost 286.0 of NE080279 [1 in = 25.4 mm].

OUTER				INNER			
STATION	TYPE OF JOINT	FAULT (in.)	WIDTH (in.)	STATION	TYPE OF JOINT	FAULT (in.)	WIDTH (in.)
0+00	Reg Cont	0.00	0.5	0+00	Reg Cont	0.00	0.5
0+46	Reg Cont	0.00	0.4	0+46	Reg Cont	0.04	0.4
0+70	Repair Ap	-0.15	0.4	0+70	Pr Relief	0.03	0.9
0+77	Repair Lv	0.12	0.4				
0+93	Reg Cont	-0.01	0.5	0+93	Reg Cont	-0.01	0.4
1+39	Reg Cont	0.00	0.3	1+39	Reg Cont	-0.01	0.4
1+86	Reg Cont	-0.03	0.3	1+86	Reg Cont	-0.02	0.4
2+32	Reg Cont	-0.01	0.3	2+32	Reg Cont	0.00	0.5
2+79	Reg Cont	-0.02	0.4	2+79	Reg Cont	0.03	0.4
3+25	Reg Cont	0.03	0.4	3+25	Reg Cont	0.00	0.4
3+72	Reg Cont	-0.01	0.4	3+72	Reg Cont	0.02	0.5
4+18	Reg Cont	-0.02	0.4	4+18	Reg Cont	0.01	0.5
4+65	Reg Cont	-0.01	0.5	4+65	Reg Cont	0.04	0.4
5+11	Reg Cont	-0.01	0.4	5+11	Reg Cont	0.00	0.4
5+58	Reg Cont	0.03	0.4	5+58	Reg Cont	0.00	0.4
6+04	Pr Relief	-0.01	0.7	6+04	Pr Relief	0.09	0.5
6+51	Reg Cont	-0.03	0.4	6+51	Reg Cont	0.00	0.5
6+97	Reg Cont	0.02	0.4	6+97	Reg Cont	0.01	0.5
7+17	Repair Ap	0.06	0.4	7+17	Repair Ap	-0.05	0.3
7+21	Repair Lv	0.16	0.4	7+21	Repair Lv	0.06	0.5
7+44	Reg Cont	0.00	0.4	7+44	Reg Cont	-0.01	0.4
7+90	Reg Cont	-0.02	0.3	7+90	Reg Cont	0.01	0.4
8+37	Reg Cont	0.00	0.4	8+37	Reg Cont	0.00	0.4
8+83	Reg Cont	0.00	0.4	8+83	Reg Cont	0.01	0.4
9+30	Pr Relief	0.01	0.6	9+30	Pr Relief	0.02	0.6
9+76	Reg Cont	0.00	0.4	9+76	Reg Cont	-0.05	0.4
10+23	Reg Cont	0.03	0.3	10+23	Reg Cont	-0.01	0.4
10+69	Reg Cont	0.02	0.3	10+69	Reg Cont	-0.01	0.4
10+92	Repair Ap	0.11	0.4				
10+96	Repair Lv	0.16	0.5				
11+16	Reg Cont	0.01	0.4	11+16	Reg Cont	0.00	0.3
11+62	Reg Cont	-0.01	0.4	11+62	Reg Cont	0.02	0.3

By 1985, the pressure relief joints had sustained about 5 million 18-kip [80-kN] ESALs in the outer lane and 1.3 million in the inner lane while the original pavement had received more than 19 million in the outer lane and 5 million in the inner lane.

The condition surveys conducted in 1985 found low- and medium-severity transverse cracks at approximately 15-20 ft [4-6 m] intervals, although some areas showed much higher levels of deterioration. Longitudinal cracking was not found. Reactive aggregate distress was found throughout both sample units, resulting in occasional scaling and joint spalls.

Joint sealant was present in most of the original contraction joints, although incompressibles were found in the joints where it was absent. As a result, medium-severity transverse joint spalling and corner spalling were frequently found in both sample units, predominantly in the outer lane. The average faulting and joint width measurements for the original contraction joints were 0.14 in [3.6 mm] and 0.68 in [17.3 mm], respectively, in the outer lane compared to 0.02 [0.05 mm] and 0.67 in [17 mm] for the inner lane and nearly 25% of the contraction joints surveyed in the outer lane failed to meet faulting acceptance criteria. The wideness of the joints is attributed to the new joint shape factor, not the use of pressure relief joints.

An average Roughness Index of 191 (fair) was obtained in 1985 using a Mays Ride Meter. An average skid number of 33 (marginal) was obtained in 1983 using a locked-wheel trailer with ASTM E274 standard tire.

The pressure relief joints have closed to an average width of 1.3 in [33 mm] and have faulted an average of 0.16 in [4.1 mm]. The filler is still partially intact in the relief joints, but where it is absent the joints are filled with incompressibles. Low-severity spalling was found at the relief joints. Contraction joints adjacent to the relief joints have not opened much greater than the average observed width of 0.67 in [17 mm].

Table 10 provides a summary of the closure of all the relief joints on this project, as measured by the Nebraska Department of Roads. Figure 57 presents a plot of the relief joint closure over time.

The pressure relief joints that were installed in 1982 may not have been necessary on this project. Although the reactive aggregates used in the original pavement are known to cause some pavement growth, they were not the highly reactive North Platte River gravels, and pressure damage was not observed on this project prior to placement of the relief joints. Joint spalling was attributed largely to reactive aggregate spalling. In addition, any built-up pressure would have been relieved during the placement of the full-depth repairs. Much higher rates of relief joint closure were observed on other Nebraska projects (e.g., NE080189) where full-depth repairs were not placed at the same time as the relief joints.

While the average amount of relief joint closure since placement is 2.7 in [69 mm], current joint measurements indicate that most of the contraction joints on the project are at least three times as wide now as when the pavement was constructed, although some of this may be due the fact that resealing was performed on the joints.

Table 10. Monitoring of Closure of All Relief Joints on NE080382 [1 in = 25.4 mm].

	EASTBOUND			WESTBOUND		
DATE	AVG CLOSURE	TOTAL CLOSURE	WIDTH OF CUT	AVG CLOSURE	TOTAL CLOSURE	WIDTH OF CUT
MAY `82	0.27	0.27	4.30	0.42	0.42	4.41
JUN `82	0.17	0.44	4.13	0.38	0.80	4.03
AUG `82	0.54	0.98	3.59	0.86	1.66	3.17
OCT `82	+0.07	0.91	3.66	+0.07	1.59	3.24
DEC `82	+0.08	0.83	3.74	+0.11	1.48	3.35
FEB `83	0.06	0.89	3.68	+0.06	1.42	3.41
MAR `83	+0.05	0.84	3.73	+0.01	1.41	3.42
MAY `83	0.21	1.05	3.52	0.17	1.58	3.25
JUN `83	0.18	1.23	3.34	0.23	1.81	3.02
JUL `83	0.42	1.65	2.92	0.46	2.27	2.56
AUG `83	0.05	1.70	2.87	0.06	2.33	2.50
OCT `83	+0.02	1.68	2.89	+0.08	2.25	2.58
NOV `83	+0.08	1.60	2.97	+0.09	2.12	2.71
FEB `84	+0.04	1.56	3.01	0.10	2.22	2.61
APR `84	0.14	1.70	2.87	0.02	2.24	2.59
MAY `84	0.17	1.87	2.70	0.06	2.30	2.53
AUG `84	0.37	2.24	2.33	0.25	2.55	2.28
SEP `84	+0.17	2.07	2.50	+0.10	2.45	2.38
APR `85	0.19	2.26	2.31	0.09	2.54	2.29
AUG `85	0.23	2.49	2.08	0.22	2.76	2.07

Note: A (+) indicates opening of the relief joint.

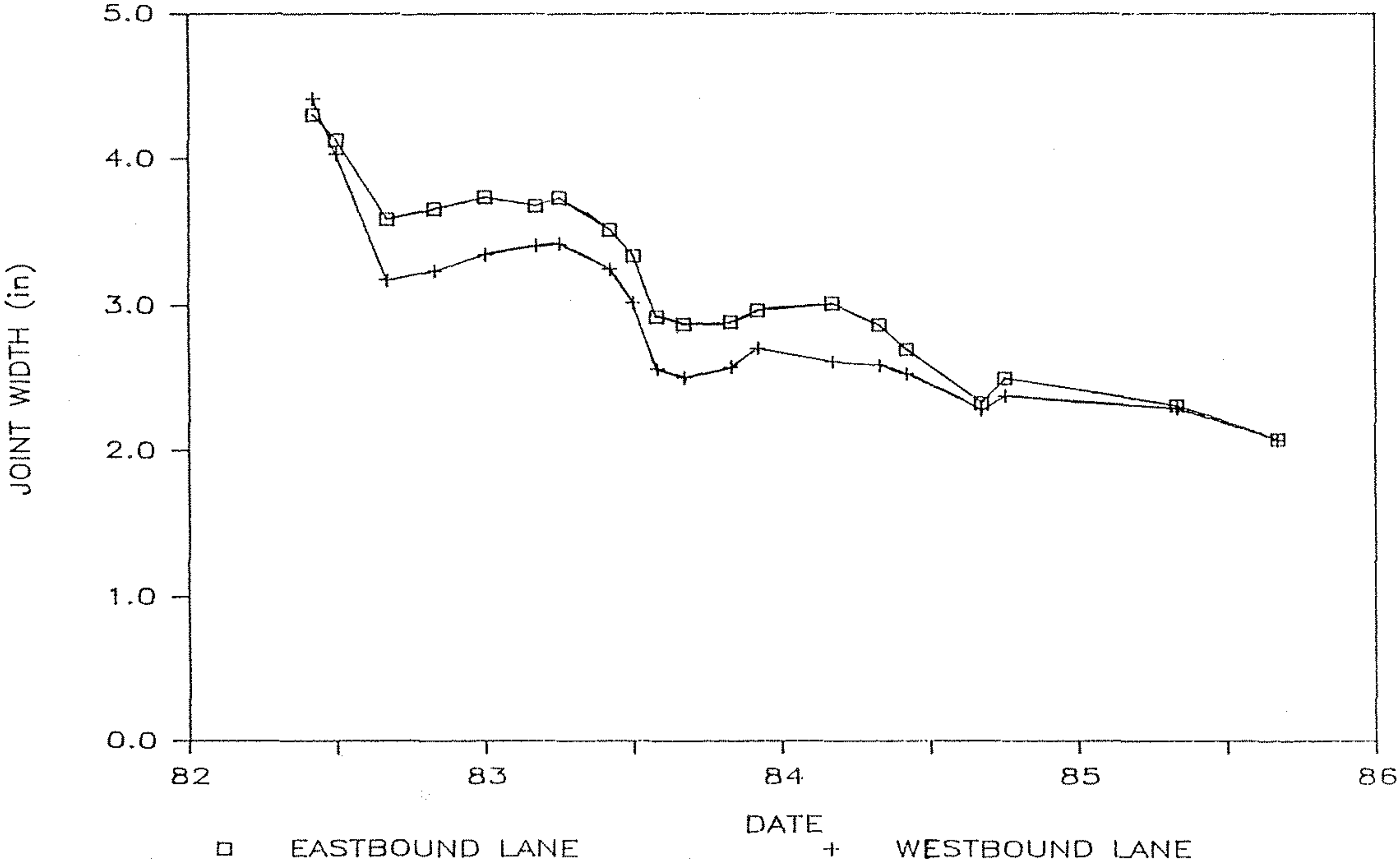


Figure 57. Pressure Relief Joint Closure Over Time on NE080382 [1 in = 25.4 mm].

Although the use of relief joints on this project may be questionable, their presence has apparently not adversely affected the project, because the faulting and opening of adjacent contraction joints is about average and nearby slabs exhibit typical types and amounts of distress. The faulting of the relief joints might have been reduced slightly if a load transfer system (e.g., dowels or a sleeper slab) had been provided.

Overall, this rehabilitation project is considered moderately successful because the full- and partial-depth repairs have reduced the roughness caused by reactive aggregate spalling at the pavement joints, although the full-depth repairs have become somewhat rough themselves. The resealed joints have also performed adequately, but are probably approaching the end of the sealant life. The relief joints are performing well, although their appropriateness is questionable.

While the original pavement has several years of serviceable life remaining, the full-depth repairs placed in 1982 should be replaced immediately. The joints will require resealing within the next few years and a longer-life sealant is recommended.

VA044000 - The original pavement was 9-in [230 mm] JRCP with 61.5-ft [18.7 m] contraction joints and was constructed and opened to traffic in 1967. Four-inch [102-mm] wide pressure relief joints were installed near mid-slab in 1976 and again in 1984 at 1000-ft [305-m] intervals using diamond saw cuts to prevent pressure buildup problems and blowups. A preformed cellular foam filler conforming to ASTM D3204 was used. Several other rehabilitation techniques were also applied to this project at various times. Full-depth repairs without mechanical load transfer devices were placed at about 31% of the transverse joints in 1984 to address transverse joint spalling and faulting and additional repairs were to be placed in 1985. A 0.5-in [13-mm] preformed bituminous expansion joint filler was provided at one joint of each of these repairs. Partial-depth repairs utilizing a calcium-aluminate cement were placed to address localized spalls in 1976 and additional repairs using a Type III Portland cement concrete were constructed in 1984. Joint resealing programs were also conducted using various sealant materials in 1973, 1976 and 1984 and were also scheduled for 1985. The design and performance of these other rehabilitation techniques is described in detail elsewhere in this report.

By 1985, the 1976 pressure relief joints had sustained about 3.8 million 18-kip [80-kN] ESALs in the outer lane while the original pavement had received about 4.3 million.

Three sample units exhibiting very different performances were surveyed in 1985. The first contained many partial-depth repairs and a few low-severity transverse cracks. The second contained a few partial-depth repairs and many low- and medium-severity transverse cracks at approximately 20-ft [6-m] intervals. The third contained full-depth repairs at nearly all of the original joints and only a few medium-severity transverse cracks. All sample units contained a pressure relief joint.

The joint sealant was intact in all of the original contraction joints that were sealed with preformed compression seals (first and second sample units), although incompressibles were frequently found in these joints.

these joints. The transverse and longitudinal joints sealed with the hot-poured sealant were still sealed and performing well. Most of the remaining original contraction joints exhibited low- and occasionally medium-severity transverse joint spalling.

The original contraction joints had faulted an average of only 0.03 in [0.8 mm] and were all acceptably smooth. A Roughness Index of 109.8 (good) was obtained in the eastbound truck lane, and a Roughness Index of 116.4 (good) was obtained in the westbound truck lane in 1980 using a Mays Ride Meter. Surface friction and PSI values were not available.

The pressure relief joints that were installed in 1976 (sample units 01 and 02) have closed from their original 4.0 in [102 mm] width to an average width of 0.8 in [20 mm] and have faulted an average of 0.11 in [2.8 mm]. The pressure relief joint that was installed in 1984 in sample unit 3 has closed from 4.0 in [102 mm] to 3.0 in [76 mm] and has faulted an 0.09 in [2.3 mm]. The filler in all of these joints is still intact. The relief joints did not exhibit any spalling problems.

The pressure relief joints installed both in 1976 and 1984 appear to have been inappropriate for this project. Those joints were placed to prevent pressure buildup problems and blowups, but there did not seem to be any signs to substantiate the need for relief joints.

While the relief joints placed in 1976 have closed more than 3 in [76 mm] in nine years, nearby original contraction joints have opened to widths of one inch [25.4 mm] or more (see Table 11), reducing the effectiveness of the preformed joint seals and encouraging the introduction of moisture and incompressibles into the pavement system.

The relief joints placed in 1984 were placed at the same time as the full-depth repairs, which also incorporated expansion approach joints. As a result, very little closure has taken place and these relief joints would probably have been unnecessary even if pressure problems had existed.

Overall, this rehabilitation project is considered successful. Although the pressure relief joints were probably unnecessary, the joint sealant maintenance program and the full- and partial-depth repairs have prevented the pavement from deteriorating rapidly.

Improved rehabilitation performance might have been achieved through the following steps:

1. Elimination of the pressure relief joints.

2. Use of low modulus adhesive sealants in contraction joints located in the vicinity of pressure relief joints.

3. Better hot-poured sealant installation (to prevent extrusion).

4. Inclusion of mechanical load transfer devices in the full-depth repairs to reduce faulting.

5. Diamond grinding after restoration to eliminate construction roughness (such as overfilled repairs).

Table 11. Summary of Joint Faults and Widths at Milepost 11.0 (Westbound) of VA044000 [1 in = 25.4 mm].

OUTER				INNER			
STATION	TYPE OF JOINT	FAULT (in.)	WIDTH (in.)	STATION	TYPE OF JOINT	FAULT (in.)	WIDTH (in.)
0+00	Reg Cont	0.02	0.7	0+00	Reg Cont	0.00	0.7
0+61	Reg Cont	-0.19	0.8	0+61	Reg Cont	0.04	1.0
1+23	Reg Cont	0.04	1.5	1+23	Reg Cont	0.00	1.2
1+84	Reg Cont	0.04	1.2	1+84	Reg Cont	0.03	1.0
2+26	Pr Relief	0.10	0.6	2+26	Pr Relief	0.13	0.9
2+46	Reg Cont	0.00	0.8	2+46	Reg Cont	0.01	0.9
3+07	Reg Cont	-0.03	1.1	3+07	Reg Cont	0.12	1.2
3+69	Reg Cont	0.03	0.7	3+69	Reg Cont	0.03	1.0
4+30	Reg Cont	0.01	0.9	4+30	Reg Cont	0.03	0.9
4+92	Reg Cont	0.06	1.3	4+92	Reg Cont	0.06	1.2
5+53	Reg Cont	0.03	0.9	5+53	Reg Cont	0.03	0.9
6+14	Reg Cont	0.00	1.5	6+14	Reg Cont	0.01	1.0
6+76	Reg Cont	0.03	1.0	6+76	Reg Cont	0.01	0.8
7+37	Reg Cont	0.04	0.9	7+37	Reg Cont	-0.02	0.8
7+99	Reg Cont	-0.08	0.9	7+99	Reg Cont	-0.02	0.8
8+60	Reg Cont	0.03	0.9	8+60	Reg Cont	0.06	0.9
9+22	Reg Cont	0.04	0.9	9+22	Reg Cont	0.01	0.8
9+83	Reg Cont	0.00	0.9	9+83	Reg Cont	0.03	0.9
10+45	Reg Cont	0.05	0.8	10+45	Reg Cont	0.03	1.1

Note: Positive faulting measurements indicate that the approach side of the joint is higher than the leave side. Negative faulting measurements indicate that the leave side of the joint is higher than the approach side.

The three sample units surveyed exhibited very different problems and their future rehabilitation must be considered separately. The first sample unit requires little work and will perform satisfactorily for several years with good maintenance. The second sample unit exhibits many medium-severity slab cracks which can be expected to deteriorate and require major rehabilitation or a structural overlay within the next four to seven years (1.6 to 4.0 million ESALs). The third sample unit is structurally sound and requires only diamond grinding and joint sealant maintenance to perform adequately for several more years.

VA064202 - The original pavement was 9-in [230 mm] JRCP with 61.5-ft [18.7 m] contraction joints and was constructed and opened to traffic in 1965. Four-inch [102-mm] wide pressure relief joints were installed near mid-slab in 1982 at 1000-ft [305-m] intervals using diamond saw cuts to prevent pressure buildup problems and blowups. A preformed cellular foam filler conforming to ASTM D3204 was used. Several other rehabilitation techniques were also applied to this project at various times. Full-depth repairs were placed in 1976 and 1984 to address transverse joint spalling (caused largely by metal joint forming inserts used in the original pavement) and faulting. Approximately 23% of the original joints had been replaced by 1985. The 1976 repairs were undercut and utilized a calcium-aluminate cement while the 1984 repairs were constructed using Type III Portland cement concrete without mechanical load transfer devices. Both sets of repairs included a preformed bituminous expansion joint filler (0.5 in [12.7 mm] thick) in the repair approach joint. Partial-depth repairs were also placed in 1976 and 1984 using similar materials. The joints were resealed in 1976 and 1984 using preformed joint seals. The design and performance of these other rehabilitation techniques are described in detail in other portions of this report.

By 1985, the pressure relief joints had sustained about 1.2 million 18-kip [80-kN] ESALs in the outer lane and 0.3 million in the inner lane while the original pavement had received about 4.4 million in the outer lane and 1.1 million in the inner lane.

The condition survey conducted in 1985 found transverse joint spalling, joint faulting, localized areas of scaling, and deteriorated joint sealant conditions. Very little transverse cracking was observed.

The preformed joint seals were still present in all of the regular contraction joints, although incompressibles were found in roughly 70% of the joints. Joint widths averaged 0.83 [21 mm] for the joints with incompressibles and 0.70 in [18 mm] for the remaining regular contraction joints. Contraction joints located near pressure relief joints and full-depth repairs were generally slightly wider than average (see Tables 12 and 13). Most of the original contraction joints exhibited little, if any, low-severity transverse joint spalling, although a few joints displayed medium-severity spalling.

The original contraction joints had faulted an average of only 0.11 in [2.8 mm] in the outer lane and 0.05 in [1.3 mm] in the inner lane. The few joints that were faulted unacceptably were the same ones that had opened exceptionally wide. The full-depth repairs generally exhibited very high amounts of faulting (0.5 in [13 mm] and greater).

Table 12. Summary of Joint Faults and Widths at Milepost 204.0 (Eastbound) of VA064202 [1 in = 25.4 mm].

OUTER				INNER			
STATION	TYPE OF JOINT	FAULT (in.)	WIDTH (in.)	STATION	TYPE OF JOINT	FAULT (in.)	WIDTH (in.)
0+00	Repair Ap	-0.14	1.1	0+00	Reg Cont	0.02	1.0
0+03	Repair Lv	0.53	0.1				
0+61	Repair Ap	0.04	1.2	0+61	Repair Ap	-0.04	1.1
0+64	Repair Lv	0.62	1.0	0+64	Repair Lv	0.27	0.7
1+23	Repair Ap	-0.18	1.3	1+23	Reg Cont	0.07	1.1
1+26	Repair Lv	0.51	0.2				
1+84	Reg Cont	0.20	1.0	1+84	Reg Cont	0.06	1.0
2+46	Reg Cont	0.17	1.1	2+46	Reg Cont	0.09	0.9
3+05	Repair Ap	0.01	0.8	3+07	Reg Cont	0.04	0.8
3+10	Repair Lv	0.08	0.7				
3+69	Reg Cont	0.02	0.9	3+69	Reg Cont	0.10	0.8
4+30	Reg Cont	0.05	0.9	4+30	Reg Cont	0.06	0.7
4+92	Reg Cont	-0.02	0.7	4+92	Reg Cont	0.10	0.6
5+53	Reg Cont	-0.03	0.8	5+53	Reg Cont	0.03	0.6
6+14	Repair Ap	-0.17	1.3	6+14	Reg Cont	0.08	0.6
6+17	Repair Lv	0.50	0.2				
6+76	Repair Ap	0.13	1.3	6+76	Reg Cont	0.17	0.6
6+79	Repair Lv	0.47	0.3				
7+37	Reg Cont	0.08	0.7	7+37	Reg Cont	0.02	0.7

Note: Positive faulting measurements indicate that the approach side of the joint is higher than the leave side. Negative faulting measurements indicate that the leave side of the joint is higher than the approach side.

Table 13. Summary of Joint Faults and Widths at Milepost 204.1 (Westbound) of VA064202 [1 in = 25.4 mm].

OUTER				INNER			
STATION	TYPE OF JOINT	FAULT (in.)	WIDTH (in.)	STATION	TYPE OF JOINT	FAULT (in.)	WIDTH (in.)
0+00	Reg Cont	0.18	0.8	0+00	Reg Cont	0.05	0.9
0+61	Reg Cont	0.21	0.8	0+61	Reg Cont	0.00	0.8
1+23	Reg Cont	0.12	0.7	1+23	Reg Cont	0.11	0.9
1+84	Reg Cont	0.18	0.8	1+84	Reg Cont	0.02	0.8
2+46	Reg Cont	0.21	0.7	2+46	Reg Cont	0.02	0.7
3+07	Reg Cont	0.30	1.2	3+07	Reg Cont	-0.03	0.8
3+69	Reg Cont	0.05	0.8	3+69	Reg Cont	0.10	0.8
4+30	Reg Cont	0.10	0.9	4+30	Reg Cont	-0.02	0.7
4+92	Reg Cont	0.18	0.8	4+92	Reg Cont	0.10	0.7
5+53	Reg Cont	0.07	0.7	5+53	Reg Cont	0.10	0.8
6+14	Reg Cont	0.08	0.8	6+14	Reg Cont	0.00	0.9
6+44	Pr Relief	0.20	2.0	6+44	Pr Relief	0.14	2.5
6+76	Reg Cont	0.19	1.2	6+76	Reg Cont	0.10	1.2
7+37	Reg Cont	0.16	0.9	7+37	Reg Cont	-0.10	0.9

Note: Positive faulting measurements indicate that the approach side of the joint is higher than the leave side. Negative faulting measurements indicate that the leave side of the joint is higher than the approach side.

Roughness and surface friction measurements were not available for this pavement section and Present Serviceability Index (PSI) could not be calculated from the information provided.

The pressure relief joints have closed to an average width of 2.0 in [51 mm] in the outer lane and 2.4 in [61 mm] in the inner lane. Faulting of the relief joints averaged 0.18 in [4.6 mm] in the outer lane and 0.16 in [3.9 mm] in the inner lane. There was no appreciable difference in closure or faulting between the relief joints surveyed in the first and third sample units (there were none in the second sample unit), in spite of the presence of full-depth repairs in the first sample unit and none in the third. The filler in all of these joints is still intact and in good condition, keeping incompressibles from infiltrating. The relief joints did not exhibit any spalling problems.

The pressure relief joints appear to have been inappropriate for this project. Those joints were placed to prevent pressure buildup problems and blowups, but there did not seem to be any signs (blowups, compression cracking, etc.) to substantiate the need for relief joints. Moreover, the placement of the full-depth (full-lane) repairs should have alleviated any pressure buildup problems in sample unit 1.

While all of the original contraction joints surveyed have opened wider than their resawed width, it cannot be determined for certain that the pressure relief joints caused this opening to any greater degree than did the full-depth repairs with integral expansion joints (the average contraction joint width in sample unit 2, which featured full-depth repairs but no relief joints, was 0.82 in [21 mm], compared to 0.93 in [24 mm] in sample unit 3, which featured a relief joint and no repairs). In any case, wider contraction joints were generally more severely faulted as well, so the relief joints were probably detrimental to overall pavement performance. This may be especially true in light of the good joint seal maintenance program in place for this section of pavement, which would have reduced pavement growth due to intrusion of incompressibles. The opening of these joints now, for whatever reason, may reduce the effectiveness of the resealing program.

Overall, this rehabilitation project is considered only partially successful. It was demonstrated that partial-depth repairs could provide good long-term (up to nine years or more) solutions to shallow joint spalling problems. Partial-width full-depth repairs were also used successfully to address these same problems. However, the full-width full-depth repairs placed in 1984 have faulted excessively due to the use of an inappropriate design, and the unnecessary use of pressure relief joints and expansion joints in full-depth repairs has caused the preformed joint seals to lose their effectiveness.

Rehabilitation is needed <u>now</u> to return this pavement to a more serviceable state and prevent accelerated deterioration. This rehabilitation should include load transfer restoration (dowels in slots or improved shear devices) at the full-depth repair joints, diamond grinding to reduce pavement roughness, and cleaning and resealing the joints with an appropriate sealant.

Considering the current pavement condition and traffic levels, the expected life of this pavement could be extended many years by implementing the above recommendations. Similar activities will eventually be necessary to address the pavement faulting and joint spalling which may develop in the next few years if the pavement is not rehabilitated soon.

VA064279 - The original pavement was a 9-in [230 mm] JRCP with 61.5-ft [18.7 m] contraction joints, constructed and opened to traffic in 1967. Four-inch [102-mm] wide pressure relief joints were installed near mid-slab in 1981 at 1000-ft [305-m] intervals using diamond saw cuts to prevent pressure buildup problems and blowups. A preformed cellular foam filler conforming to ASTM D3204 was used. Several other rehabilitation techniques were also applied to this project at various times. Full-depth repairs were placed in 1981 at approximately 81% of the joints to address transverse joint spalling (caused largely by metal joint forming inserts used in the original pavement) and faulting. The repairs included a preformed bituminous expansion joint filler (0.5 in [12.7 mm] thick) in the repair approach joint and utilized no mechanical load transfer devices. Joint resealing was also accomplished in 1981 using a hot-poured sealant. Diamond grinding was accomplished in the eastbound lanes in 1984 to reduce pavement roughness. The full-depth repairs and joint resealing programs are described in detail elsewhere in this report.

By 1985, the pressure relief joints had sustained about 2.8 million 18-kip [80 kN] ESALs in the outer lane and 1.6 million in the center lane while the original pavement had received about 6.7 million in the outer lane and 3.4 million in the inner lane.

The condition survey conducted in 1985 found low-severity transverse joints, joint faulting, and localized areas of scaling. Very little transverse cracking was observed, particularly in the eastbound lanes.

All of the regular contraction joints were still well sealed and incompressibles were not found in the joints in either sample unit surveyed. The joints that had been diamond ground were smoother than the ones that had not been ground. The average faulting was 0.03 in [0.76 mm] for the diamond ground center lane joints and 0.12 in [3.0 mm] for the unground center lane joints and similar numbers were obtained from the outer lane joints. If the faulting immediately after grinding was zero, the measurements indicate that faulting is continuing to develop at a rate of approximately 0.03 in [0.8 mm] per year or 0.08 in [2.0 mm] per million 18-kip [80-kN] ESALs in the center lane. Since faulting in the unground sample unit developed over a period of 18 years and 3.4 million ESALs, it can be suggested that center lane faulting has developed at a rate of less than 0.01 in [0.25 mm] per year or 0.04 in [1.0 mm] per million ESALs. Thus, it appears that the <u>rate</u> of development of faulting has increased.

Roughness was measured in early 1985 using a Mays Ride Meter:

LANE	ROUGHNESS
Eastbound outer (ground)	72 (excellent)
Eastbound center (ground)	66 (excellent)
Eastbound inner (ground)	64 (excellent)
Westbound outer (not ground)	145 (fair)

Surface friction measurements were not available. Present Serviceability Index (PSI) cannot be calculated due to insufficient information.

The pressure relief joints have closed from their original 4.0 in [102 mm] width to an average width of 1.15 in [29 mm], and have faulted an average of 0.15 in [3.8 mm]. The outer lane relief joints had closed to an average width of 1.47 in [37 mm], while the middle lane relief joints had closed to an average width of 0.83 in [21 mm]. Thus, more closure took place in the middle lane where fewer full-depth repairs were placed. Relief joint faulting averaged about 0.15 in [3.8 mm] and was considered acceptable. Figure 58 shows a typical relief joint on this project.

The filler in the relief joints was still intact and keeping incompressibles from infiltrating, although a few of the pressure relief joints exhibited medium-severity transverse joint and corner spalling.

The pressure relief joints installed in 1981 were inappropriate for this project. Those joints were placed to prevent pressure buildup problems, but there appeared to be no signs (blowups, compression cracks, etc.) to substantiate a need for relief joints. Moreover, the installation of the full-depth repairs should have alleviated any built-up pressure.

Because most of the surveyed original transverse contraction joints had been replaced by full-depth repairs, it is difficult to determine what effect the relief joints had on the adjacent contraction joints. However, as previously noted, it is clear that the installation of the full-depth repairs had the effect of relieving some of the built-up pressure, as reflected by less closure of relief joints in the lanes which had a higher density of full-depth repairs. The closure that did take place must be due to a slight opening of the adjacent contraction and repair joints. Since a good sealing program has been in place, it might be assumed that the joints are being filled with pumped material from below. With so much expansion capacity available (in the relief joints and repairs) it is not surprising that the slabs are moving and pumping to fill this capacity.

Overall, this rehabilitation project is considered unsuccessful because: 1) the full-depth repairs that were installed to address joint spalling roughness have become rough due to faulting and have, in fact, faulted more than twice as much in four years as the original contraction joints have in 18 years; 2) the pressure relief joints were probably unnecessary and have allowed the original and repair joints to open and pump more readily; 3) faulting measurements indicate that the diamond grinding reduced roughness only temporarily and that significant roughness is redeveloping only one year later; and 4) the transverse drains that were installed with some of the full-depth repairs have apparently been ineffective in reducing pumping and faulting.

Suggestions for improvements include the incorporation of load transfer devices in future full-depth repairs, installation of subdrains along the entire project, cement grout undersealing at least in the vicinity of the repairs and known voids, and regrinding the pavement when roughness becomes unacceptable. Removal of the relief joints could also be considered.

Considering the current pavement condition, traffic levels and rates of deterioration, the expected life of this pavement (to a state of

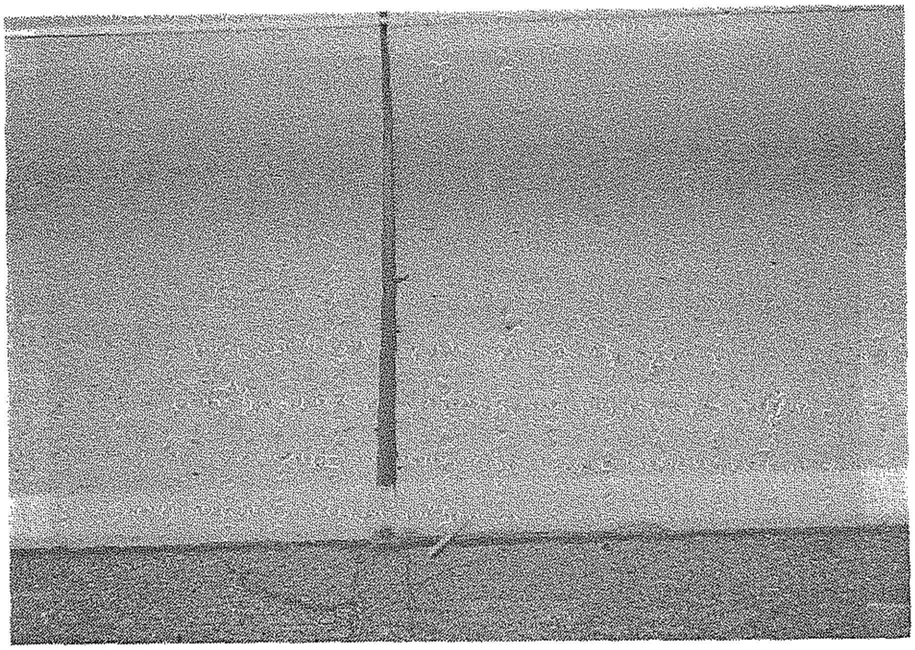


Figure 58. Photo of Pressure Relief Joint in Sample Unit 2 (Station 5+86) of VA064279 (Diamond-Ground Direction).

unacceptable roughness) is approximately 2 to 4 years from the date of survey (1.8 to 3.6 million ESALs in the outer lane). The unground section would benefit from more immediate attention. With proper rehabilitation, this project should provide many additional years of service because it shows little evidence of fatigue or structural failure.

VA064284 - The original pavement was a 9-in [230 mm] JRCP with 61.5-ft [18.7 m] contraction joints, constructed and opened to traffic in 1968. Four-inch [102-mm] wide pressure relief joints were installed near mid-slab in 1978 and 1984 at 1000-ft [305-m] intervals using diamond saw cuts to prevent pressure buildup problems and blowups. A preformed cellular foam filler conforming to ASTM D3204 was used. Several other rehabilitation techniques were also applied to this project at various times. Full-depth repairs were placed at most of the original joints in 1978 (eastbound lanes) and 1984 (westbound lanes) to address transverse joint spalling (caused largely by the use of metal joint forming inserts in the original pavement) and faulting. The older repairs in the eastbound lanes were scheduled for replacement in 1985. The 1978 repairs included flat anchor bars for load transfer. The 1984 repairs included a preformed bituminous expansion joint filler (0.5 in [12.7 mm] thick) in the repair approach joint and utilized no mechanical load transfer devices. Partial-depth repairs were placed in 1973 and 1978 at localized joint spalls. Joint resealing programs were accomplished in 1973, 1978 and 1984 using a variety of sealant types. The designs and performances of the full-depth repairs are described in detail elsewhere in this report.

By 1985, the 1978 pressure relief joints had sustained about 4.3 million 18-kip [80-kN] ESALs and the 1984 relief joints had received about 0.8 million while the original pavement had received about 7.4 million.

The condition survey conducted in 1985 found low-severity transverse joint and corner spalls, joint faulting, and localized areas of scaling and deteriorated joint sealants. Only two low-severity transverse cracks were observed in the surveyed sample units.

Roughness was measured in 1985 using a Mays Ride Meter. A Roughness Index of 133 (fair) was obtained for the eastbound driving lane (which featured the 1978 repairs), and a Roughness Index of 144 (fair) was obtained for the westbound driving lane (which featured the 1984 repairs). Surface friction measurements and data for Present Serviceability Index (PSI) computations were not available.

The 1978 pressure relief joints have closed from their original 4.0 in [102 mm] width to an average width of 0.7 in [18 mm], which is about the same width as the surrounding repair and contraction joints. They have faulted an average of 0.19 in [4.8 mm]. The filler in these joints was frequently absent and the joints were full of incompressibles. The only 1984 pressure relief joint surveyed is still 4.0 in [102 mm] wide, and has faulted 0.15 in [3.8 mm]. There was no filler in this joint, nor is it believed that a filler was ever placed in this joint. None of the pressure relief joints have exhibited significant spalling problems.

The pressure relief joints installed in 1978 were inappropriate due to the large number of full-depth repairs placed that same year. The removal of deteriorated concrete for full-depth repairs relieves built-up compressive stresses in a pavement. However, transverse joints located near the relief joints are not significantly wider or more faulted than other transverse joints and the slabs themselves are still uncracked. Thus the use of these relief joints appears to have done no harm and may have prevented some spalling of the 1978 full-depth repairs, although it is known that the calcium aluminate repairs are susceptible to spalling.

The pressure relief joints installed in 1984 were inappropriate for the section of pavement surveyed, as indicated by the fact that the one joint surveyed had not closed at all one year later. These lanes had already been relieved through the installation of full-depth repairs with expansion joints on one side at every original contraction joint. The repair joints have not been adversely affected by all of the expansion capacity of the pavement, but they have been in place only one year.

Overall, this rehabilitation project is not considered completely successful, primarily due to the poor performance of the full-depth repairs. The 1978 repairs are currently in poor condition and are apparently about to be replaced. The 1984 repairs are displaying some unusually high faults for such new repairs.

This project also illustrates the type of conditions for which pressure relief joints may be appropriate for a pavement. These conditions include such items as long joint spacing, no intermediate slab cracking, low truck traffic volumes, and a mild environmental region.

VA081148 - The original pavement was 9-in [230 mm] JRCP with 61.5-ft [18.7 m] contraction joints and was constructed and opened to traffic in 1965. Four-inch [102-mm] wide pressure relief joints were installed near mid-slab in 1976 at 1000-ft [305-m] intervals using diamond saw cuts to prevent pressure buildup problems and blowups. A preformed cellular foam filler conforming to ASTM D3204 was used. Several other rehabilitation techniques were also applied to this project at various times, including a 1984 CPR demonstration project in the first surveyed sample unit. The CPR project included the installation of full-depth undercut repairs at major cracks and deteriorated joints, partial-depth repairs utilizing various materials, pozzolan-cement grout undersealing of each joint, installation of six-in [150-mm] perforated pipe longitudinal underdrains, load transfer restoration with shear devices and retrofit dowels, diamond grinding and joint resealing. The second sample unit contained pressure relief joints, cement grout undersealing and joint resealing. The designs and performances of the full- and partial-depth repairs and joint resealing programs are described in detail in other portions of this report.

By 1985, the 1976 pressure relief joints had sustained about 6.6 million 18-kip [80-kN] ESALs in the outer lane and 1.3 million in the inner lane while the original pavement had received about 8.8 million in the outer lane and 1.2 million in the inner lane.

The condition survey of the second sample unit conducted in 1985 found several panels that exhibited no cracking at all followed by several more panels that displayed numerous transverse cracks of varying severity.

These different performances are separated by a construction joint that marks the end of one day's work and the beginning of another.

The regular contraction joints in sample unit 2 had been resealed with preformed compression seals which were displaying adhesion failure and contained incompressibles. Joint widths in this area averaged 0.57 in [14 mm]. A few joints exhibited medium-severity spalling. These joints had not been diamond ground and average faulting measurements of 0.07 in and 0.02 in [1.8 and 0.5 mm] were obtained from the outer and inner lanes, respectively.

A Roughness Index of 156 (fair) was obtained in the southbound truck lane in 1985 using a Mays Ride Meter. Surface friction and Present Serviceability Index (PSI) values were not available.

The pressure relief joint installed in sample unit 2 in 1976 has closed to 1.0 in [25 mm] in the outer lane and 0.7 in [18 mm] in the inner lane. Adjacent contraction joints do not seem to have opened or faulted significantly. Relief joint faulting measured 0.49 in [12 mm] in the outer lane and 0.21 in [5 mm] in the inner lane.

The filler in the relief joint is still intact and in good condition, keeping incompressibles from infiltrating. The relief joint did not exhibit any spalling problems.

Other relief joints observed in the same area have closed and faulted similarly and their adjacent contraction joints have opened. Faults of as much as 2 in [51 mm] have been reported at some relief joints.

The pressure relief joints installed in 1976 appear to have been inappropriate for this project. Those joints were placed to prevent pressure buildup problems and blowups, but there did not seem to be any signs (blowups, compression cracking, etc.) to substantiate the need for relief joints. Moreover, the placement of the full-lane full-depth repairs that incorporated expansion joints should have alleviated any pressure buildup problems.

The inclusion of relief joints on this project did not appear to adversely affect the pavement performance (except for a moderate increase in roughness due to faulting across the relief joint itself). Adjacent transverse contraction joints seemed to be no wider than more distant joints, indicating that the relief joints did not contribute significantly to the preformed joint sealant failures.

This rehabilitation project is considered successful because the overall rate and level of deterioration has been reduced by the applied rehabilitation. While it is possible that localized areas of roughness will develop near the pressure relief joints and some of the full-depth repairs, it is expected that this pavement will provide generally good serviceability for 5 to 7 years from the date of survey (4 to 6 million ESALs) given the current pavement condition and traffic levels.

Portions of the project (including sample unit 2) were scheduled for CPR in 1985.

CHAPTER IV

PANEL CONCLUSIONS AND RECOMMENDATIONS

PERTAINING TO THE DESIGN AND USE OF PRESSURE RELIEF JOINTS

Conclusions

1. The use of pressure relief joints was unwarranted on most of the projects surveyed and often caused more distress than would have been prevented. Where pressure buildup problems/potential existed that were not related to reactive aggregates, other techniques could have been considered (i.e., good joint maintenance programs) which would have been equally successful in preventing pressure damage and less likely to result in other types of pavement deterioration.

2. The use of pressure relief joints was generally warranted on projects which included reactive aggregates in the PCC.

3. The unnecessary or excessive use of pressure relief joints often results in one or more of the following:

    - Excessive opening of adjacent cracks and contraction joints, allowing the entry of incompressibles and water, which causes increased faulting and spalling and continued pavement expansion (due to entrapment of incompressibles).

    - Shearing of longitudinal joint ties and faulting of the longitudinal joint.

    - Premature failure of load transfer shear devices and of adjacent contraction joint sealant (particularly preformed compression seals).

    - Loss of load transfer and pavement support resulting in increased pumping, faulting, corner breaks and punchouts.

4. Pressure relief joints are not likely to be as detrimental to pavement performance when the existing pavement is free of working transverse cracks and only low volumes of heavy truck traffic are present (less than 100 trucks per day). Excessive opening of adjacent contraction joints may still occur.

5. The largest portion of relief joint closure occurs within the first year after installation if the use and construction of the relief joints is appropriate.

6. Where used and constructed appropriately, pressure relief joints have been found to be effective in preventing the development of pressure damage for 3 to 7 years. New or additional joints must be considered when the old ones become ineffective.

7. Relief and expansion joints near secondary structures may provide relief as far as 2000 ft [600 m] away. Contraction joints located near such features exhibited much greater widths and faults and higher incidences of sealant failure than more distant joints.

8. The installation of full-depth repairs provides relief of built-up pressure and decreases the need to install pressure relief joints in the vicinity of the repair.

9. Pressure relief joints placed without load transfer devices fault rapidly. Pressure relief joints incorporating load transfer devices have been constructed and have faulted very little under very heavy traffic.

10. The placement of joint sealant material over the relief joint filler material improves retention of the filler material.

11. Pressure relief joints are effective in preventing blowups, but do not necessarily prevent joint deterioration. They prevent the hazard and repair expense associated with the occurrence of blowups, but the resulting deterioration of the original pavement (deteriorated cracks and joints) may require more extensive repair than would have developed if relief joints had not been placed (deteriorated joints only).

12. Blowups have about the same effect as pressure relief joints on the total movements at adjacent contraction joints.

13. Wide asphalt concrete-filled pressure relief joints installed in concrete pavements that are to be overlaid can result in "humping" of the overlay over the relief joint, deterioration of adjacent cracks and joints and increased incidence and severity of reflection cracking. Larger relief joint spacings tend to produce less joint and crack deterioration and reflection cracking, but larger "humps." Shorter spacings produce little "humping" but very high densities and severities of reflection cracking, including at the relief joint.

14. Foam-filled pressure relief joints installed in concrete pavements that are to be overlaid can result in rapid deterioration of the overlay directly over the relief joint because the soft filler material provides no support to the overlay.

15. Skewing overlaid pressure relief joints has no apparent effect on the magnitude of humping that develops. The pavement may be slightly more acceptable to the user since only one wheel will cross the joint at a time.

16. Overlays placed over pressure relief joints that have been in place for several years may perform well without exhibiting much of the deterioration normally associated with overlaid relief joints because much of the pavement movement will have already taken place.

17. Illinois DOT's "heavy-duty" pressure relief joints have performed well under heavy traffic (greater than 10 million ESAL). Excessive rutting produces unacceptable roughness at the transition between the overlay and the PCC surface.

Recommendations

1. In general, the installation of pressure relief joints is recommended only where reactive aggregates are present and a pressure buildup problem exists or where an asphalt concrete overlay is to be placed over a Portland cement concrete pavement that is expected to develop pressure buildup problems, as described previously.

2. The installation of pressure relief joints in continuously reinforced concrete pavement (CRCP) is not recommended. Pressure relief joints placed in CRCP may cause tight adjacent cracks to deteriorate in a short time (i.e., less than two years) even under relatively light traffic and in the presence of a stabilized subbase.

3. The installation of pressure relief joints is not recommended for pavements with short joint spacing except for protection of secondary structures. Their use may result in decreased load transfer and pavement support, and increased occurrence and severity of associated distresses.

4. The continued use of pressure relief and expansion joints to protect bridges is recommended for all pavement types. Adjacent contraction joint sealant reservoirs should be designed (or maintained) and sealed to accommodate potentially large movements (i.e., use low-modulus sealants in properly designed reservoirs). It is recommended that load transfer be established for 6-10 contraction joints from the pressure relief joint for plain undowelled pavements.

5. Pressure relief joint placement (where appropriate) must consider the rate of pavement growth and the location and effectiveness of other pressure-relieving features such as concurrently or recently placed full-depth repairs and existing relief and expansion joints near secondary structures. The minimum distance to such full-depth repairs should consider their design (i.e., inclusion of expansion joints in the repair) and the length of time the slab removal gap was left open prior to concrete placement. New pressure relief joints should be placed at least 1000 feet [305 m] from active pressure relieving features. These considerations also apply to pavements to be overlaid.

6. Most new pressure relief joint widths should be limited to 1 to 2 in [25 - 51 mm] (maximum) to reduce the possibility and severity of over-relieving the pavement. Pavements with reactive aggregates may require greater relief joint widths or more frequent relief joint installations. Good performance has been obtained on highly expansive pavements with pressure relief joint spacings of 2000 ft [600 m].

7. Sealant caps should be placed over the relief joint filler material in all narrow relief joints to help keep the filler in the slot and keep incompressibles from infiltrating. The sealant cap should be recessed appropriately so that it doesn't extrude as the pavement closes.

8. Deep cleaning of joints and cracks with high-pressure water to remove trapped incompressibles followed by joint resealing should be tried on an experimental basis to relieve pressure buildup caused by entrapment of incompressibles.

9. An alternative to the installation of undowelled pressure relief joints is the removal and replacement of small slab sections (small full-depth repairs across the pavement) utilizing dowels in the repair joints to reduce the possibility and severity of pumping and faulting at the joints. Pressure relief might be accomplished by either leaving the repair open for 24 hours or by incorporating a narrow (1-in [25-mm]) expansion joint at one repair joint.

10. An alternate approach to pressure relief in CRCP that could be tried experimentally is to remove and replace small sections of the pavement (at typical relief joint intervals), leaving the repair hole open for 24 hours or more to allow the pavement to expand slightly before reestablishing the reinforcing steel through the repair and placing new concrete.

11. Where pressure relief joints are to be incorporated in full-depth repairs, the total expansion capacity should be limited to 1 in [25 mm] per 1000 ft [305 m] because removing the deteriorated material provides some pressure relief.

12. The design of adjacent contraction joints (reservoir dimensions and sealant material properties) should be checked prior to the installation of pressure relief joints to insure that the expected movement of the joint will not cause the sealant to fail. The determination of expected joint movements should include consideration of relief joint spacing, location of other pressure relieving features (bridge expansion joints, recently-placed full-depth repairs, etc.), location of the contraction joints on vertical curves, ambient temperature range, documented joint movement on similar projects and other important factors. If anticipated contraction joint movements will cause sealant failure, the joints should be widened and/or more extensible sealants should be installed. Pressure relief joints are not recommended to be installed on projects with preformed compression seals unless resealing of the regular contraction joints (with an extensible sealant) is scheduled to be performed concurrently.

13. Where blowups have occurred recently (not due to expansive or reactive aggregate), thus relieving pressure, joint cleaning and resealing should be considered as an alternative to pressure relief joint installation.

14. The use of pressure relief joints in pavements that are about to be overlaid is not generally recommended. Reflective cracking will rapidly develop over the relief joints and adjacent joints and cracks, requiring extra maintenance attention.

15. It is occasionally desirable to provide pressure relief joints in structurally sound PCC pavements prior to placing asphalt concrete overlays. Candidate projects include those with long joint spacings and joints filled with incompressibles when the built-up pressure has not been relieved by blowups, repairs or other pressure relieving features. Construction of an AC overlay may increase the pavement moisture to the point where hydro-thermal expansive pressures exceed concrete strength, resulting in blowups beneath the overlay. The use

of pressure relief joints may also be appropriate if the presence of reactive aggregates will cause pavement expansion and blowups beneath the overlay. Pressure relief joint designs for these applications should include the same considerations described previously.

16. Asphalt concrete is recommended for relief joint filler material in relief joints that are constructed in PCC pavements which will be overlaid.

17. On an experimental basis, dowels placed in slots across a pressure relief joint may be tried to evaluate the performance and cost-effectiveness of providing load transfer across such a joint. A recommended procedure for establishing load transfer with dowels placed in slots is given in Reference 146.

CHAPTER V

FULL-DEPTH REPAIR BACKGROUND INFORMATION

Full-Depth Repairs/Joint Reconstruction

Introduction

Jointed concrete pavements deteriorate at joints and at intermediate cracks. Although most of the deterioration occurs at joints for a variety of reasons (25, 29), often intermediate cracks deteriorate (i.e., spall and fault) under repeated heavy traffic loadings. This usually occurs when doweled joints become "frozen" and the intermediate cracks are forced to accommodate all horizontal slab movement. The cracks open slightly, aggregate interlock is rapidly lost, the cracks begin to fault, and eventually the steel ruptures. This leads to further faulting and spalling of the crack. Some projects will exhibit joints with very little deterioration, but one or more intermediate working cracks in each slab that are acting as joints. Working cracks should be repaired with either a tied or dowelled full-depth repair (25).

Repairing Large Areas

In some situations the existing distress is so extensive that the repairing of every deteriorated joint and crack is impractical. Repair costs can sometimes be reduced by simply removing and replacing larger areas of the concrete slab. This is referred to as "slab replacement" and generally provides more reliable performance than numerous small repairs. A separate pay item is often included in contracts for this type of repair because the unit repair cost per square yard is significantly less than for smaller full-depth repairs.

Determining Repair Boundaries

Repair boundaries must be selected so that all significant underlying distress is removed along with the distress that appears on the pavement surface. Deterioration near the joints is often more extensive at the bottom of the slab than at the top, especially in freeze-thaw climates. This is illustrated in Figure 59. Coring studies can provide information concerning the extent of the deterioration beneath the slab surface. When coring of the pavement is infeasible, chains, ball peen hammers, or lengths of reinforcing bar may be used to "sound out" the concrete. Areas issuing a clear ringing sound are judged to be sound concrete while those emitting a dull sound are considered to be weakened and should be marked for removal. Sophisticated equipment for detecting unsound concrete is also commercially available (147, 148, 149).

Repair boundaries should not be located too close to existing transverse cracks or joints, or adjacent slab distress may occur. Repair boundaries should be placed no closer than 3 ft [0.9 m] from tight, non-working cracks containing reinforcing steel and 6 ft [1.8 m] from working cracks (30).

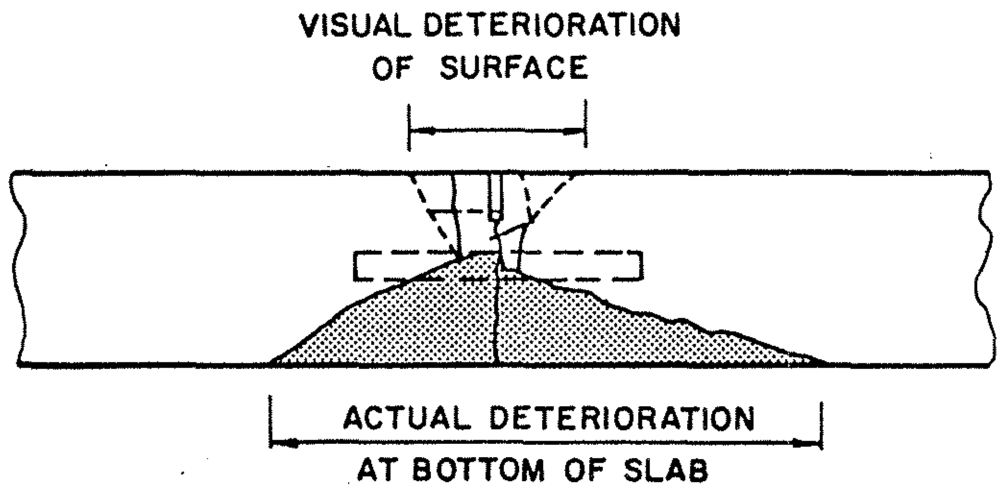


Figure 59. Illustration of Potential Extent of Deterioration Beneath Joint (30).

Establishing minimum repair sizes reduces or eliminates rocking, pumping and breakup, and facilitates construction work (i.e., drilling of holes for dowels) within the patch boundaries. Very short repairs have been known to crack longitudinally, resulting in patch failure similar to edge punchout failure in continuously reinforced pavements. They may also deflect independently of the adjacent slabs and cause excessive pumping. Recommended minimum repair dimensions are given below (32).

1. Repairs that are tied, dowelled, or undercut for load transfer should have a minimum length of 6 ft [1.8 m] (112).

2. Repairs that rely solely on aggregate interlock for load transfer should be not less than 6 ft [1.8 m] long in low traffic areas and 8 to 10 ft [2.4 to 3.0 m] long in medium-high traffic areas.

3. Repairs that extend across the full lane width are recommended.

Full-depth repairs may be located on either side of a joint, or on both sides, depending on the location of the distresses. Repair boundaries may be either skewed or perpendicular to the pavement edge and are generally placed to match the existing joints. However, it is often difficult to install dowels parallel to the direction of traffic in skewed joints.

Design of Repair Joints

The repair joint design is crucial to the performance of the repair. Joint design still depends largely upon engineering experience, although excellent analytical techniques for calculating edge and joint stresses and deformations are available.

Types of Joint Faces

The two types of sawed transverse joint faces used in full-depth patches are rough-faced and smooth-faced joints.

Rough-faced joints are constructed using a diamond blade concrete saw to cut through less than 30% of the slab depth and breaking out the area to be patched. The rough face provides some aggregate interlock properties for load transfer (see Figure 60). Supplemental mechanical load transfer devices may also be provided. Special care must be taken during the breakout operation to avoid undercutting or damaging the surrounding slab area. This procedure is no longer recommended by many agencies.

Smooth-faced joints are produced by sawing through most of the slab depth (a full-depth cut is often used). The smooth face provides essentially no aggregate interlock (see Figure 60) and load transfer must be provided by some mechanical device (e.g., dowel bars, tie bars, or shear device). This technique is now recommended by most state highway agencies.

Expansion or pressure relief joints placed between the slab and the repair often produce poor results. Their poor load transfer capabilities result in higher repair and slab stresses and deflections.

Types of Load Transfer

Load transfer refers to the ability of a joint or crack to transfer load (and deflection) across the joint or crack and is often defined as the ratio of the deflection of the unloaded side to the deflection of the

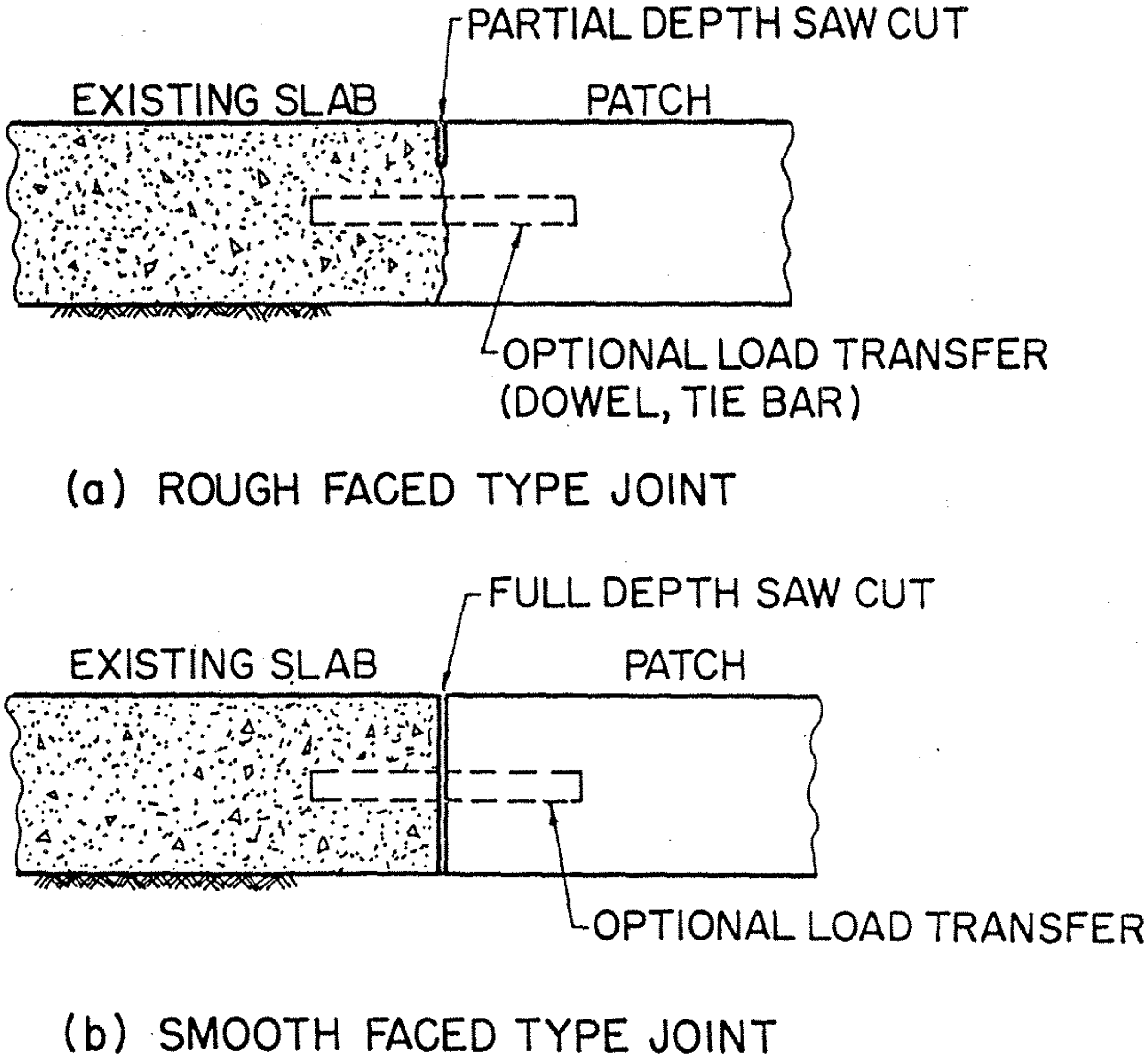


Figure 60. Rough- and Smooth-Faced Joints (30).

loaded side. Poor load transfer usually leads to serious spalling, patch rocking, faulting, and corner breaks. Common techniques used to develop load transfer across a transverse repair joint include:

1. Tie Bars. Deformed rebars are grouted into the existing slab. This is typically accomplished by drilling holes on 12 in [305 mm] centers into the exposed face of the existing slab. A nonuniform spacing of three to five rebars (clustered in the wheel paths) may be most efficient. A quick-setting, nonshrinking mortar or a high-viscosity epoxy can be used to anchor the deformed rebar into the existing slabs. Tie bars should be used only where no movement of the joint is desired.

2. Dowel Bars. Smooth steel bars (dowels) may be inserted into holes drilled into the existing slab, as described above. Dowels are often coated with a lubricant to facilitate horizontal movement at the joint (e.g., due to thermal expansion and contraction) and should be coated with epoxy, plastic or some other material to resist corrosion. Dowels should be used when horizontal movement of the joint is desired. Figure 61 shows a typical layout for a given full depth repair. The performance of dowelled repair joints has been inconsistent due to difficulties in securing the dowels in the existing slab. Weakening of the concrete surrounding the dowel (from drilling the dowel holes) and ineffective grouting procedures have been common problems.

3. Repair Under-Cutting. The subbase/subgrade may be excavated from beneath the slab and filled with concrete. This type of repair is often referred to as an "inverted tee" repair. This method is not recommended for areas with frost heave problems unless it is used in conjunction with tie or dowel bars. Even in non-frost areas, poor load transfer will be obtained if good concrete consolidation is not achieved in the lip of the repair or if the repair settles.

4. Aggregate Interlock. This type of load transfer is obtained from the mechanical interlock of repair material with rough-faced joints. It should only be used in conjunction with short joint spacings and is unreliable, especially in cold weather or when pressure relief joints are located nearby and the joints open.

Contraction joints may be reestablished within the repair boundaries when it is infeasible to construct them at the repair boundaries. The repair is tied to the adjacent slabs with deformed rebar. The contraction joint is then formed using prefabricated dowel and fiberboard assemblies (see Figure 62), dowels mounted on chairs and a partial-depth saw cut, or other appropriate means.

Selecting the Proper Joint Design

The joint design is often selected based on the load transfer required to prevent serious repair faulting or rocking for a given climate, traffic and foundation. Truck traffic affects the load transfer required. The following general recommendations have been used to determine load transfer required in some states:

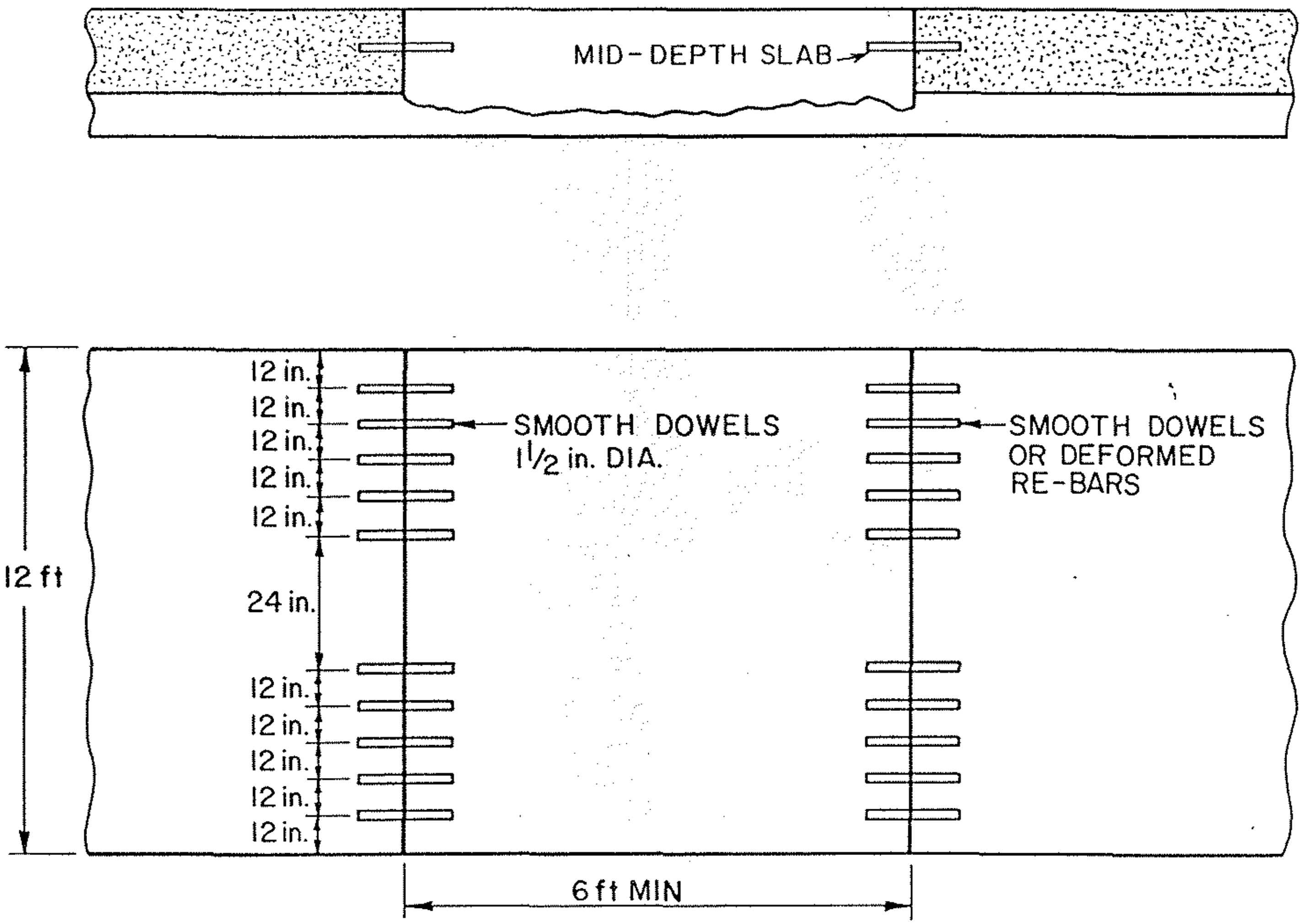


Figure 61. Layout of Dowels for Full-Depth Repairs in Illinois (25) [1 in = 25.4 mm].

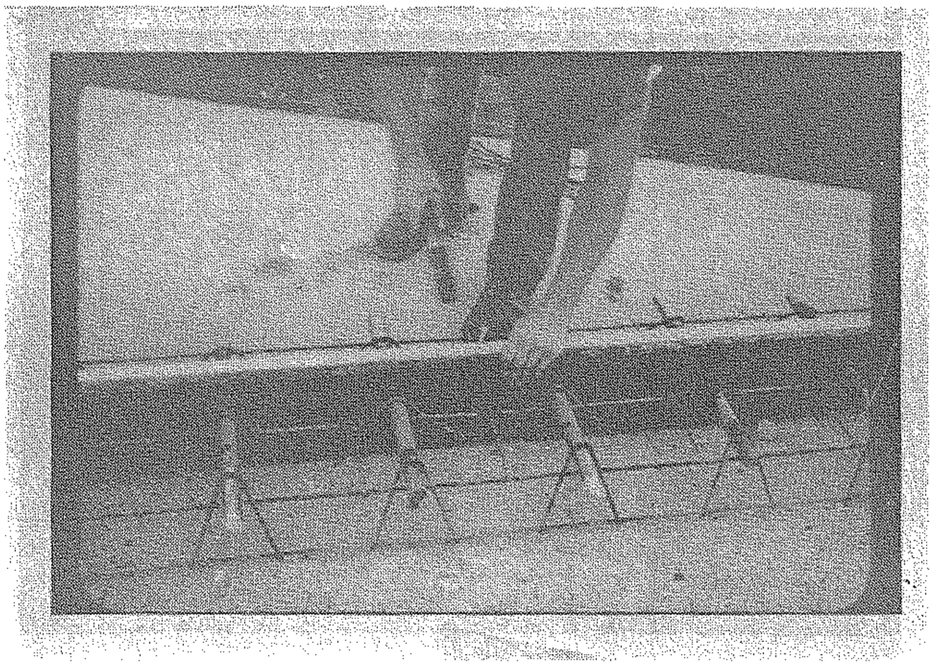


Figure 62. Photo of Prefabricated Joint Assembly from Michigan.

1. Jointed Plain Concrete Pavements. The repair should be dowelled if the existing slab contains dowels at the transverse joints. If the existing slab does not contain dowels, aggregate interlock is often used, although the guidelines given in Figure 63 may provide superior performance.

2. Jointed Reinforced Concrete Pavements. Because long-jointed pavements are usually subject to large thermal joint movements, dowels are normally installed at both repair joints or at one joint with tie bars at the other. Joint design (dowel and tie bar arrangements) should be similar to that provided in new pavements. Tied "inverted-tee" repairs have provided satisfactory performance on some projects.

Materials Considerations

Conventional Repair Materials

The concrete mixture is normally selected based on the curing time available before the repair must be opened to traffic. Regular concrete paving mixtures are often used when curing times of several days are possible. If earlier opening times (e.g., 1 to 3 days) are required, Type III (high early strength) cement, increased cement content, cement with accelerators, minimal mixing water (decreased water:cement ratio), and/or insulating layers placed on top of the repair to retain the heat of hydration have been used to produce high early strengths. Typical repair mixtures utilize seven to nine bags of cement per cubic yard of concrete and a water reducer or set-accelerator to achieve one- to three-day openings (and some as short as 4 hours) (25, 31, 97, 114, 130). Table 14 provides some approximate minimum times for opening concrete repairs to traffic for various slab thicknesses, mix designs, ambient temperatures, admixtures, and curing conditions (31).

Proprietary Repair Materials

Rapid-setting materials are available for very early openings and are summarized in Reference 98. Costs of many of these special materials are much greater than for typical paving concrete and for this reason they are not commonly used for full-depth repairs.

Precast Full-Depth Repairs

Projects requiring rapid repair can utilize precast repairs. Full-depth precast permanent repairs have been used experimentally in several states (Michigan, California, Florida, Virginia, etc.) to provide repair of deteriorated pavement joints (30, 36, 44, 45, 77, 112, 114, 131). The Michigan DOT proved that a concrete pavement could be repaired with a precast slab within one and one-half hours (114) if mechanical load transfer devices were not used. Cost for such a repair is somewhat higher than for conventional full-depth repair.

Since there is no load transfer capability inherent in the use of precast repairs, it is necessary to subsequently install some form of mechanical load transfer device if load transfer is desired.

Several techniques have been tried for precasting the slabs, removing the damaged concrete and placing the precast slab. These are detailed in Reference 114.

CLIMATE **	SUBBASE	ADTT * LIGHT	MEDIUM	HEAVY
Wet	Granular	Aggregate Interlock	Dowels***	Dowels
	High Quality Stabilized	Aggregate Interlock	Aggregate Interlock***	Dowels
Dry	Granular	Aggregate Interlock	Aggregate Interlock***	Dowels
	High Quality Stabilized	Aggregate Interlock	Aggregate Interlock	Aggregate Interlock***

* Average Annual Daily Truck Traffic in a Given Lane (excluding pickups and panels). - Heavy > 1500, Medium 300 to 1500, Light < 100

** The Boundary Between Wet and Dry Climates is the Zero Thornthwaite Index Contour

*** Dowels are Recommended if Future Pavement Life Greater than 5 Years is Required.

Figure 63. Guidelines for Load Transfer Requirements for Full-Depth Repairs.

Table 14. Early Opening Guidelines for Full-Depth Repairs [1 in = 25.4 mm; °C = (°F-32)5/9] (Ref. 31).

Slab Thickness (inches)	Ambient Temperature At Placement (°F)	Full-Depth Repair Mixtures/Curing* (hours after placement) A	B	C	D	E	F
7	40	203	90	69	29	28	7
	50	125	60	41	21	20	5
	60	80	45	28	17	16	4
	70	60	38	21	14	13	3
	80	48	35	17	13	11	3
	90	40	30	13	13	9	3
8	40	145	59	55	24	24	6
	50	82	40	35	18	17	5
	60	58	31	24	13	13	4
	70	42	26	17	11	10	3
	80	35	23	13	10	9	3
	90	29	22	11	9	8	3
9	40	82	34	37	15	16	5
	50	51	25	23	12	13	3
	60	28	19	16	9	9	3
	70	25	16	12	8	7	3
	80	20	14	10	6	6	3
	90	17	12	8	5	5	3
10	40	45	18	23	9	9	3
	50	30	14	14	7	7	3
	60	20	10	9	5	5	3
	70	15	9	7	4	4	3
	80	12	7	5	4	4	3
	90	9	6	4	3	3	3

*All mixtures contain 650 pounds cement per cubic yard [386 kg per cubic meter] and 2% CaCl.

<u>Mixture Characteristics</u>:	<u>A</u>	<u>B</u>	<u>C</u>	<u>D</u>	<u>E</u>	<u>F</u>
water/cement ratio	0.42	0.42	0.35	0.42	0.35	0.35
cement type	I	I	I	III	I	III
superplasticizer	no	no	yes	no	yes	yes
fiberglass insulation	no	yes	no	yes	yes	yes

Note: These results are based on research done at the University of Illinois, Department of Civil Engineering, using a computer program written in the Microsoft BASIC language. They are intended as guidelines and should only be used after careful evaluation.

Construction Considerations

Standard Repair Considerations

The major construction steps involved in full-depth repair and joint reconstruction of jointed concrete pavements are:

1. Sawing the repair boundaries.

2. Removing concrete within the boundaries.

3. Repairing the slab foundation (or excavating for undercutting).

4. Placement of load transfer devices (if used).

5. Placement of reinforcement (if used).

6. Placement of the concrete.

7. Curing and opening to traffic.

Techniques for all of the above steps and construction procedures and guide specifications are included in the Appendices.

Reconstructed or reformed transverse joints are constructed similar to new construction. They are resawed (if necessary to obtain a proper sealant reservoir shape factor) and sealed with a preformed seal or a high-quality poured sealant with the appropriate shape factor.

Multiple-Lane Repair Operations

On multiple-lane facilities, deterioration may occur across two or more lanes. If the distress occurs in only one lane, it is not necessary to repair the other lane(s). When two or more adjacent lanes contain distress and they must be repaired one lane at a time to maintain traffic flow, the best results are usually obtained by repairing the lane with the least truck traffic first. This places any cracks that may form in a location where they will be least likely to deteriorate under future traffic (72).

If blowups occur due to concentration of expansion forces in one lane (while the other is being repaired), it may be necessary to cut pressure relief joints at intervals of 600 to 1200 ft [183 to 366 mm] or delay repair until cooler weather occurs.

Past Performance of Full-Depth Repairs/Joint Reconstruction

The performance of full-depth repairs has been inconsistent. While there are many documented cases of repairs that have performed satisfactorily, the performance record of many in-service full-depth repairs has been poor (25, 72, 77, 84, 97, 112, 119, 128, 131). Failures due to repair settlement, faulting, loss of load transfer and subbase support, spalling, pumping and frost heave have been observed within a few months after construction on some repairs. Thus, there is considerable room for improvement in the design and construction of full-depth repairs.

CHAPTER VI

EVALUATION OF IN-SERVICE FULL-DEPTH REPAIR INSTALLATIONS

AZ010255 - The original pavement was a 12-in [305 mm] JPCP with 15-ft [4.7 m] contraction joints and was constructed and opened to traffic in 1960. A third lane was added to the inside of the existing pavement lanes in 1982. Full-depth repairs (in the form of full-slab replacements) were placed in 1982 at approximately 2% of the panels to address slab cracking. Other rehabilitation techniques applied include partial-depth repairs, diamond grinding, and tied concrete shoulders. Due to high traffic volumes, only the outer lane was surveyed.

The following table summarizes faulting of the surveyed joints [1 in = 25.4 mm]:

	REG CONT outer lane	REP APP outer lane	REP LV outer lane
Total Meeting Acceptance Criteria	100%	100%	100%
Average Fault	0.01	0.02	0.02
ESALs Sustained By Joint Type (millions)	6.2	1.2	1.2

The acceptance criteria includes all faults in the range of -0.20 in to 0.20 in [-5.1 mm to 5.1 mm]. Faults in this range are considereded acceptable by the user.

The design of the full-depth repairs is shown in Figure 64. The two slab replacements are performing very well. No distresses were observed with the repairs, and they are exhibiting minimal faulting.

The transverse contraction joints associated with the full-depth repairs are also performing well. Joint widths ranged from 0.5 in to 3.4 in [13 mm to 86 mm]. The large joint widths may be attributed to the full-depth repairs' proximity to a bridge, where considerable movement is taking place. The joint sealant in the repair joints was still intact, and no incompressibles were present. Spalling was not a problem at any of the repair locations.

The full-depth repairs placed in 1982 have successfully addressed the joint spalling and faulting of the original pavement. They are performing well and show no signs of distress. Since only two repairs were evaluated, it is difficult to assess the effectiveness of this repair technique. Based on their condition, it would appear that they have been successful. However, there presently exists a large number of cracked slabs. It is possible that not enough full-depth repairs were placed in 1982.

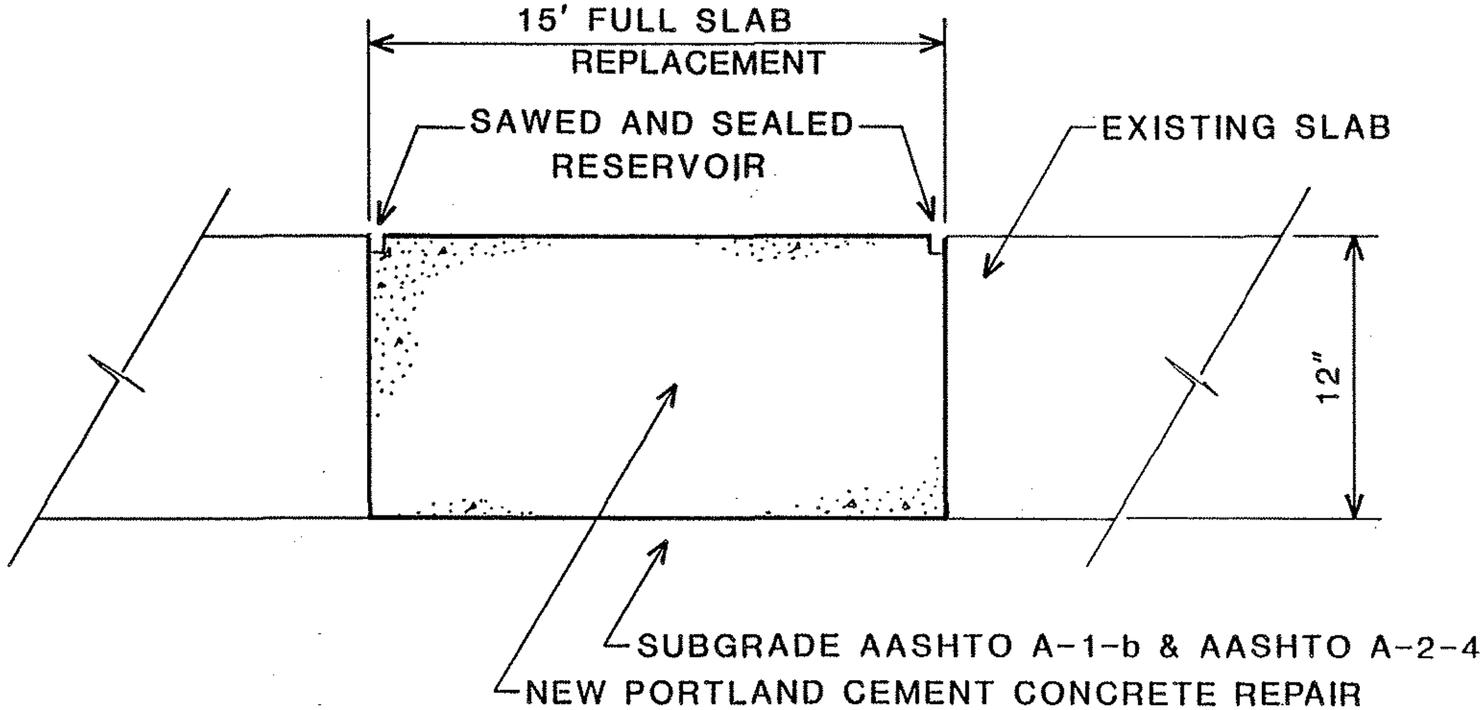


Figure 64. Illustration of Full-Depth Repair Design Used on AZ010255 [1 in = 25.4 mm].

Overall, this rehabilitation project is considered successful because of the combined performance of the full-depth repairs, partial-depth repairs, and diamond grinding. Most of the joints in both lanes are smooth, and spalling is not a major problem. There are several locations (particularly sample unit 2, stations 4+60 to 5+40 and stations 6+80 to 7+60) where longitudinal cracking exists over many consecutive slabs and is in need of immediate repair. Also, selected slabs throughout the sample units are displaying medium-severity slab cracking. Thus, additional slab replacements should be performed now.

In nearly 26 years of service, this pavement has performed rather well. While diamond grinding was performed to address roughness problems, the slabs are still structurally sound. The environment in which the pavement exists has probably contributed to its performance, as this pavement is not subjected to the freeze-thaw action or moisture excesses that are so detrimental to pavements in the northern parts of the country.

IL055098 - The original pavement was a 10-in [254 mm] JRCP with 100-ft [30.5 m] contraction joints and was constructed and opened to traffic in 1963. Full-depth repairs were placed at most joints in 1983 to address a moderate to severe "D" cracking problem. However, the following winter many of the repairs developed deep spalling and pumping due to loss of load transfer, and approximately 50% of the 1983 repairs were replaced in 1984. The 1983 repairs were typically 4 ft [1.2 m] wide and used a 3-2 dowel configuration (3 dowels in the outer wheelpath and 2 in the inner wheelpath). Dowel bars were 1.25 in [32 mm] in diameter and 18 in [457 mm] long. Several of the 1983 repairs were partial lane width. The 1984 repairs were typically 6 ft [1.8 ft] wide and contained the same dowel bars and configuration as the 1983 repairs. Some 1984 repairs also incorporated a 2-in [51 mm] wide preformed cellular plastic joint filler on both the approach and leave sides of the repair. Other rehabilitation techniques applied to the pavement include pressure relief joints, cement grout undersealing, and longitudinal underdrains. The design of the full-depth repairs is shown is Figure 65.

The following table summarizes faulting of the surveyed joints [1 in = 25.4 mm]:

	1983 REPAIRS				1984 REPAIRS			
	REP APP		REP LV		REP APP		REP LV	
	outer	inner	outer	inner	outer	inner	outer	inner
Total Meeting Acceptance Criteria	73%	100%	100%	77%	83%	91%	92%	82%
Average Fault	0.10	0.10	0.08	0.18	0.12	0.07	0.11	0.14
ESALs Sustained By Joint Type (millions)	2.5	0.6	2.5	0.6	1.3	0.3	1.3	0.3

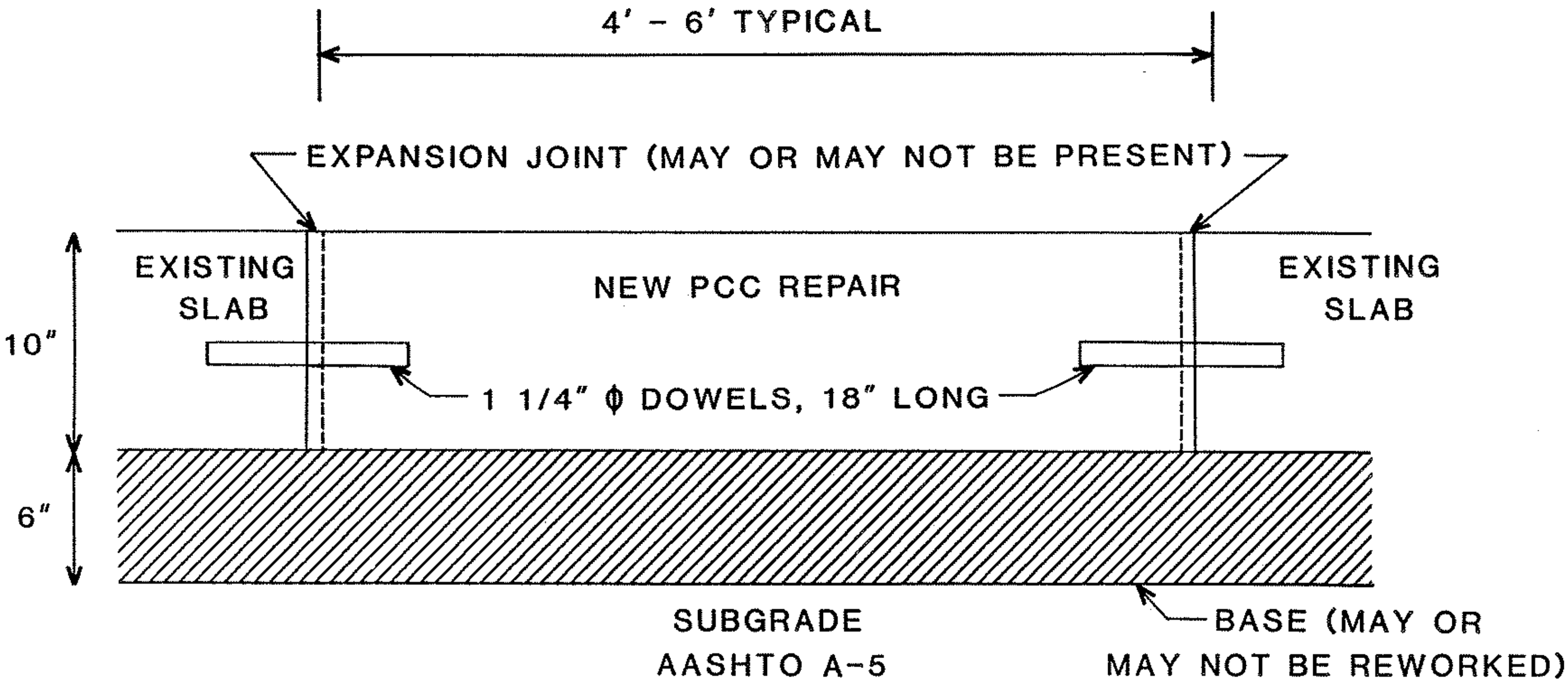


Figure 65. Illustration of Full-Depth Repair Design Used on IL055098 [1 in = 25.4 mm].

Both sets of repairs do not always meet the acceptance criteria for smoothness, which is somewhat surprising for the short period for which they have been down. This large faulting may be due to poor load transfer at the repair joints. The loss of load transfer may be attributed to heavy truck traffic, poor support of the dowels due to a relatively large annular gap around the dowels, and inadequate grouting techniques. No difference in faulting is detected between the 1983 and 1984 repairs.

Joint spalling was present in both repairs and is attributed to the loss of load transfer, the absence of a properly formed sealant reservoir, and the presence of incompressibles in the joints. Slightly worse spalling was observed at the 1983 repairs than at the 1984 repairs (see Figure 66). Some corner spalls were observed (for both repairs) where repair boundaries in adjacent lanes were not collinear and the concrete between the offset transverse joints spalled. Spalls were also observed in the 1983 repairs where "D"-cracked concrete was apparently not entirely removed prior to placement of the full-depth repairs. The repairs themselves did not exhibit any "D" cracking.

The majority of the repairs surveyed exhibited little or no cracking and the few cracks that were observed were in the 1983 repairs and were of low severity. Longitudinal joint spalling was observed within a series of five repairs where the longitudinal joint was not sawed or formed.

The location of the worst spalling (repair side of approach joint) indicates that the repair is being forced backwards against the approach joint by the torque of the passing wheels. The tight approach joints and open leave joints at some of the repairs support this theory.

Most of the full-depth repair transverse contraction joints associated with the full-depth repairs are performing poorly, as evidenced by the spalling and faulting conditions observed. Contraction joint widths ranged from 0.0 in to 0.3 in [0.0 mm to 7.6 mm] and most (84% of the 1983 repairs, 72% of the 1984 repairs) were tight. Joint width measurements (and faulting) were consistently larger for repair <u>leave</u> joints than for repair <u>approach</u> joints.

The partial-lane-width full-depth repairs are performing poorly, as they are exhibiting high faults, large spalls, and tight joints. In addition, "D" cracking is present on the original slab outside of the repair.

The full-depth repairs placed in 1984 addressed spalling (due to "D" cracking) and faulting (due to pumping) of the original joints. These distresses were due primarily to loss of load transfer at the 1983 repairs, as discussed earlier. Also, some of the 1984 repairs were directed at areas that should have been repaired in 1983 but were not, due to limited funds. It is possible that earlier placement of these repairs would have reduced the subsequent deterioration of the pavement and number of repairs needed, and extended the remaining serviceable life of the section.

Overall, this rehabilitation project is not considered completely successful, primarily due to poor performance of the full-depth repairs. The expected life of this pavement is approximately five to seven years

1983 REPAIRS

REPAIR WIDTH ⟷

L=100%	38%	100%	92%	TRAFFIC →
M=0%	62%	0%	8%	
H=0%	0%	0%	0%	
100%	75%	100%	92%	TRAFFIC →
0%	17%	0%	8%	
0%	8%	0%	0%	

1984 REPAIRS

REPAIR WIDTH ⟷

L=100%	64%	100%	100%	TRAFFIC →
M=0%	36%	0%	0%	
H=0%	0%	0%	0%	
92%	75%	100%	100%	TRAFFIC →
8%	25%	0%	0%	
0%	0%	0%	0%	

NOTE: All undamaged joints and all joints with low severity spalling were rated as low severity.

Figure 66. Summary of Spalling at Repair Locations on IL055098.

from the installation of the full-depth repairs (6.5 to 9.1 million ESALs). Thus major rehabilitation, in the form of additional repairs and/or an overlay, is expected to be needed as early as 1988 or as late as 1990. However, in anticipation of increased routine maintenance expenditures associated with continued pavement deterioration due to "D" cracking, IDOT has planned an overlay for the project for the fall of 1985.

IL055102 - The original pavement was a 10-in [254 mm] JRCP with 100-ft [30.5 m] contraction joints and was constructed and opened to traffic in 1963. Rehabilitation techniques applied include full- and partial-depth repairs, longitudinal underdrains, cement grout undersealing, and diamond grinding. Typically 4-ft [1.2 m] wide patches (many only partial lane width) were placed in 1983 at approximately 20% of the joints to address joint spalling. The repair contained 1.25-in [32 mm] dowels in a 3-2 dowel bar configuration (3 dowels in the outer wheelpath and 2 dowels in the inner wheelpath). The typical repair design is similar to that used on IL055098 (as shown in Figure 65).

The following table summarizes faulting of the surveyed joints [1 in = 25.4 mm]:

	REG CONT		REP APP		REP LV	
	outer	inner	outer	inner	outer	inner
Total Meeting Acceptance Criteria	100%	100%	100%	100%	50%	100%
Average Fault	0.03	0.02	0.08	0.06	0.21	0.04
ESALs Sustained By Joint Type (millions)	13.0	2.0	2.6	0.4	2.6	0.4

The effect of the diamond grinding is reflected in the faulting measurements. The repair leave joint faulting is larger than the repair approach joint faulting.

There are some low- to medium-severity joint spalls associated with the repairs (in the repair itself and in the adjacent slabs), but nothing that greatly affects repair performance. Some of the smaller joint spalls may have been caused by the diamond grinding operation. Spalling was observed on the repair side of the approach joint, which often exhibited medium-severity spalling. Medium-severity corner spalling was evident throughout both sample units, located primarily on the approach side of the repair. Longitudinal joint spalling of low to medium severity was observed along the outside edge (between adjacent lanes) of several repairs.

The full-depth repairs placed in 1983 were directed at the joint spalling and corner breaks of the original pavement. These repairs were placed before the diamond grinding, which provided the maximum benefit from the grinding. The full-depth repair/diamond grinding combination has thus far successfully addressed the problem of original joint faulting and spalling. Because grinding was performed soon after full-depth repair installation, the faulting observed can be concluded to have developed since the placement of repairs (i.e., grinding removed only construction roughness and immediate settlement, not long-term faulting).

Although some of the repairs are exhibiting medium-severity spalling on the approach side of the repair, they are considered to be performing well because they are still providing a smooth riding surface.

Overall, this rehabilitation project is considered successful, primarily due to the smooth surface provided by the rehabilitation. The expected life of this pavement is approximately six to nine years from the time of the diamond grinding operation (6 to 9 million ESALs). Thus major rehabilitation, in the form of additional full-depth repairs and diamond grinding or an overlay, is expected to be needed as early as 1990 or as late as 1993, depending on how quickly the faulting redevelops.

Although constructed and opened at the same time as IL055098, which is just adjacent to it, this project (IL055102) has performed much better. The primary reason for this is that this original construction section utilized an aggregate which was less susceptible to "D" cracking than that used on IL055098. The "D" cracking problem has been a major source of deterioration for IL055098. The comparison of these two projects illustrates the effect of materials on pavement performance. In addition, IL055098 has received slightly more truck traffic than IL055102, which has also contributed to its deterioration.

IL280-74 - The original pavement was a 10-in [254 mm] JRCP with 100-ft [30.5 m] contraction joints and was constructed and opened to traffic in 1962. The pavement design, construction, and rehabilitation history is the same for both Interstate sections, the only difference being that I-280 carries more traffic. Full-depth repairs were placed in 1984 at approximately 48% of the joints to address joint spalling and faulting. The repairs contained 1.25-in [32 mm] dowels in a 3-3 dowel bar configuration (3 dowel bars in each wheelpath). The transverse repair joints were sawed and sealed after placement. The typical repair design used on this project is shown in Figure 67. Other rehabilitation techniques applied include partial-depth repairs, undersealing, diamond grinding, longitudinal underdrains, and joint resealing.

The following table summarizes faulting of the surveyed joints [1 in = 25.4 mm]:

	I-280 REPAIRS				I-74 REPAIRS			
	REP APP		REP LV		REP APP		REP LV	
	outer	inner	outer	inner	outer	inner	outer	inner
Total Meeting Acceptance Criteria	100%	100%	100%	100%	100%	100%	100%	100%
Average Fault	-0.01	0.02	0.05	0.04	-0.01	0.01	0.02	0.02
ESALs Sustained By Joint Type (millions)	1.0	0.2	1.0	0.2	0.6	0.1	0.6	0.1

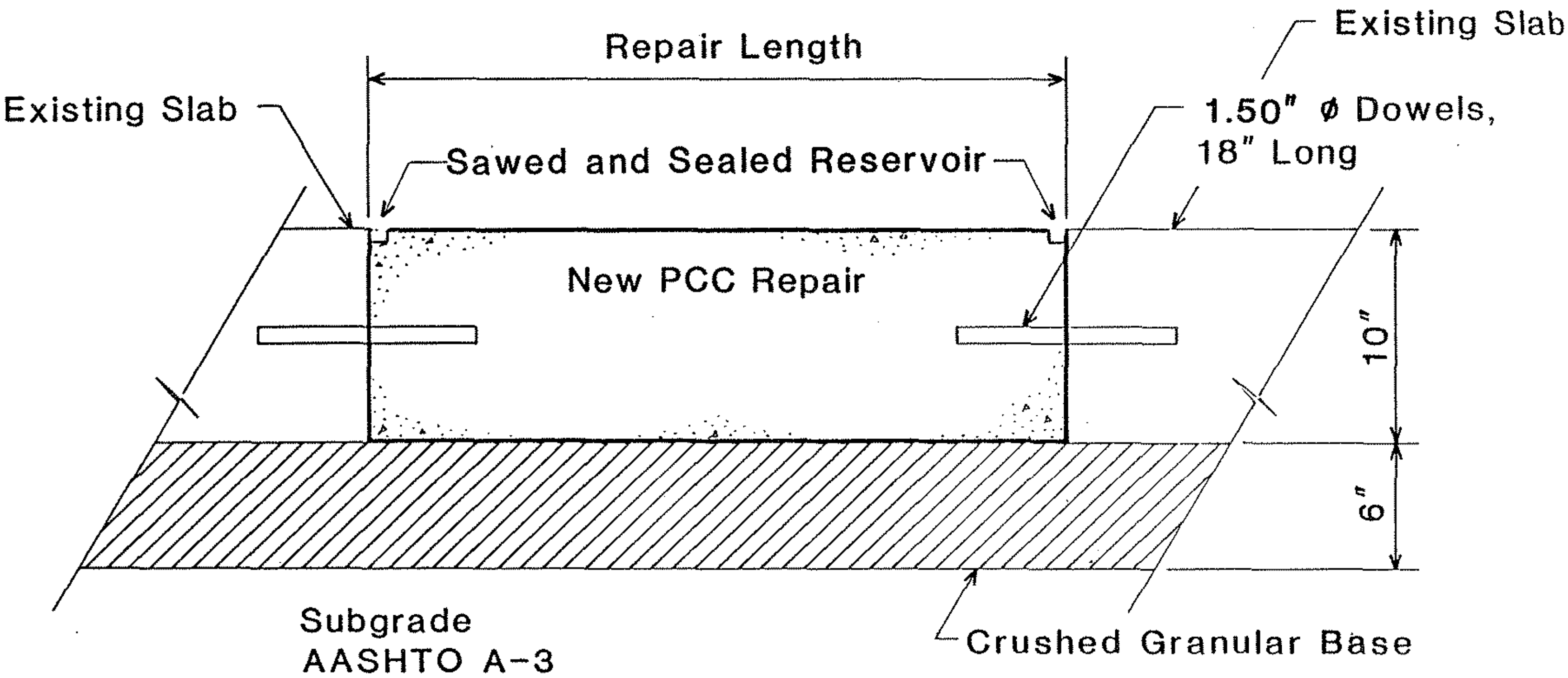


Figure 67. Illustration of Full-Depth Repair Design Used on IL280-74
[1 in = 25.4 mm].

Faulting of the repair joints was not a problem. All of the repair joints were in the smooth ride range of -0.20 in to 0.20 in [-5.1 mm to 5.1 mm], which can be attributed to diamond grinding. Repair joint faults are generally not much greater than faults at the original contraction joints, although one repair leave joint fault measured 0.14 in [3.6 mm]. Repair leave joints were commonly faulted more than repair approach joints, and the repair joints on I-280 were faulted slightly more than those on I-74 (due to there being more traffic on I-280). Although the faults are still very small, there is clearly a tendency towards tilted (approach end low) repairs, which typically accompanies pumping.

While the repair joints are generally smooth, the sealant is present and performing well in only 53% of the joints. The sealant is partially or completely absent from the remaining 47% of the repair joints. The repair joint widths ranged from 0.0 in to 1.0 in [0 mm to 25 mm]. The joints with smaller widths (0.0 in to 0.2 in [0 mm to 5 mm]) were frequently the ones whose sealant was absent.

Low- and medium-severity spalling was found near several of the surveyed full-depth repairs. Figure 68 summarizes the location and severity of the observed spalling at repair locations.

The majority of the surveyed full-depth repairs were placed in the inner lane, which may be an indication that climate and age, rather than traffic, are the major causes of distress.

The full-depth repairs placed in 1984 addressed the joint spalling and joint faulting of the original pavement. They are performing well after only one year of service, but do exhibit relatively high faults at the repair leave joints at a few locations and the joint sealant is absent from many of the repair joints. These two conditions <u>could</u> lead to increased faulting and joint spalling, in spite of the efforts that were apparently taken to insure good performance and serviceability (undersealing, diamond grinding, and longitudinal underdrains). It is possible that there may have been difficulties in the dowel installation (poor grouting technique, excessively large dowel holes, etc.) which might contribute to large faults at some joints.

Design/construction improvements that might have been considered would have been reconstruction of the shoulder with a deeper free-draining base and a concrete surface (to reduce edge stresses and pavement deflections), and better joint sealant installation.

Overall, this rehabilitation project is considered successful after one year of service, due to the combined performance of all the applied rehabilitation techniques.

Considering the current pavement condition and traffic levels, additional major rehabilitation (full-depth repairs, diamond grinding, structural overlays, etc.) should not be necessary for 4 to 7 years from the date of survey (4.0 to 7.0 million ESALs) for I-280 and 6 to 10 years from the date of survey (3.6 to 6.0 million ESALs) for I-74. However, as previously noted, it is possible that age and climatic effects may take a larger toll on the pavement condition than the effect of traffic, thereby requiring rehabilitation for each section sooner.

I-280

L 100% M 0% H 0%	L 100% M 0% H 0%	L 100% M 0% H 0%	L 92% M 8% H 0%	TRAFFIC →
L 100% M 0% H 0%	L 83% M 17% H 0%	L 100% M 0% H 0%	L 100% M 0% H 0%	TRAFFIC →

I-74

L 100% M 0% H 0%	L 100% M 0% H 0%	L 100% M 0% H 0%	L 100% M 0% H 0%	TRAFFIC →
L 100% M 0% H 0%	L 100% M 0% H 0%	L 100% M 0% H 0%	L 100% M 0% H 0%	TRAFFIC →

NOTE: ALL UNDAMAGED JOINTS AND ALL JOINTS WITH LOW SEVERITY SPALLING WERE RATED AS LOW SEVERITY.

Figure 68. Summary of Spalling at Repair Locations on IL280-74.

Due to the present pavement condition, it is recommended that partial-depth repair and joint resealing work be accomplished as soon as possible. However, 100-ft [30.5 m] slabs are difficult to seal properly due to the large amount of movement associated with such long slabs.

IL080105 - The original pavement was a 10-in [254 mm] JRCP with 100-ft [30.5 m] contraction joints and was constructed and opened to traffic in 1960. Full-depth repairs were placed in 1983 at several joints and at several major cracks. However, deep spalling occurred in many of the 1983 repairs and they were replaced in 1985. The 1983 repairs were typically 4 ft [1.2 m] wide and used 1.25-in [32 mm] diameter, 18-in [457] long dowels. A 3-2 dowel bar configuration was used (3 dowels in the outer wheelpath and 2 in the inner wheelpath). The repair joints were not resealed after placement. The 1985 repairs were generally 6 ft [1.8 m] wide and used 1.50-in [38 mm] diameter, 18-in [457 mm] long dowels in a 5-5 configuration (5 dowels in both the outer and inner wheelpaths). The repair joint was sawed and sealed after placement. One 1983 repair and five 1985 repairs were surveyed. The 1983 repair were surveyed in June, 1985, and the 1985 repairs were surveyed in August, 1986. The full-depth repair designs are shown in Figure 69. Other rehabilitation techniques applied include pressure relief joints, undersealing, diamond grinding, longitudinal underdrains, load transfer restoration, and joint resealing.

The following table summarizes faulting of the surveyed joints [1 in = 25.4 mm]:

	1983 REPAIRS-('85 Data)				1985 REPAIRS-('86 Data)			
	REP APP		REP LV		REP APP		REP LV	
	outer	inner	outer	inner	outer	inner	outer	inner
Total Meeting Acceptance Criteria	N/A	100%	N/A	100%	100%	100%	100%	75%
Average Fault	N/A	0.05	N/A	-0.02	-0.04	0.03	0.03	0.14
ESALs Sustained By Joint Type (millions)	N/A	0.4	N/A	0.4	1.1	0.2	1.1	0.2

It is observed that faulting of both sets of repairs was generally within the smooth ride range, but one 1985 repair displayed an unusually high leave fault (0.46 in [12 mm]). This is an unexpected fault for such a new repair which not only contains 10 dowel bars, but dowels of 1.5 in [38 mm] diameter.

The faulting of both sets of repair joints did not seem to indicate any consistent pattern in the vertical orientation of the repair. This is probably due to the fact that the 1985 repairs have only been in place for one year and have not started to show "true" faults (from pumping). The faulting that the 1985 repairs are displaying now may have been built in during construction. The faulting of the 1985 repairs is comparable to that of the 1983 repair (which had been diamond ground).

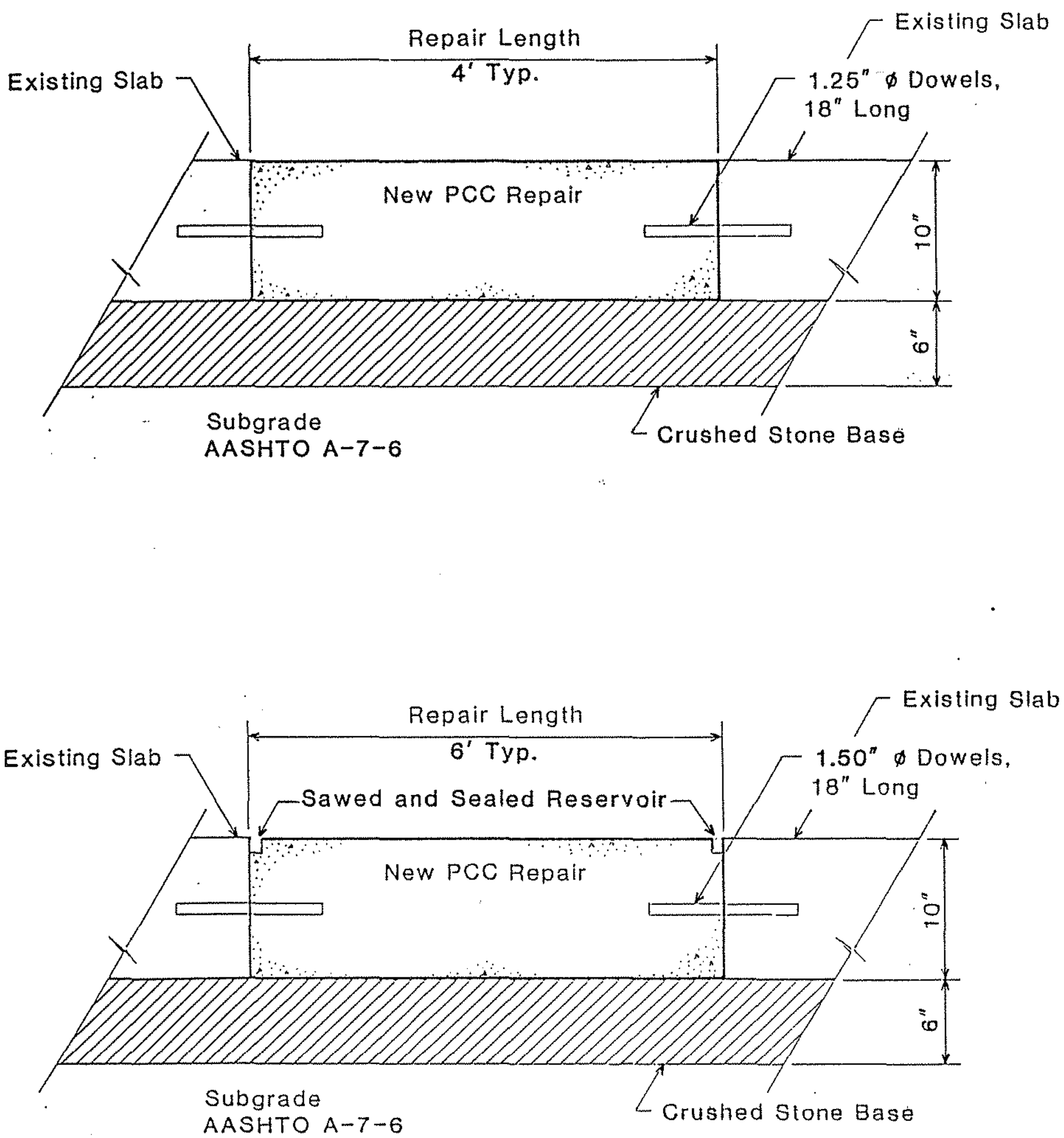


Figure 69. Illustration of Full-Depth Repair Designs Used on IL080105 [1 in = 25.4 mm].

The 1985 repair joints had all been resawed and resealed after their placement. However, the 1983 repair had not been sealed, and both repair joints contained incompressibles and, subsequently, is spalling on the repair side of each joint. In addition, the joints were tight on this repair, as compared to an average width of 0.7 in [18 mm] for both the approach and leave joints of the 1985 repairs.

This project demonstrates the importance of sawing and sealing full-depth repair joints, as the 1985 repair joints (which were sealed) are performing extremely well. The 1985 repairs are structurally sound and will be able to sustain additional traffic loading. However, the 1983 full-depth repair is experiencing spalling problems at both repair joints, due to the lack of sawing and resealing those joints.

The expected life of this pavement is approximately five to eight years (5.5 to 8.8 million ESALs) from the date of the rehabilitation. Thus, full-depth repairs and diamond grinding or an overlay is expected to be needed as early as 1989 or as late as 1992.

However, prior to that time the following measures are recommended:

1. A review of the 1983 repairs should be conducted, with removal and replacement of those exhibiting serious distress.

2. The failed shear devices should be removed and replaced.

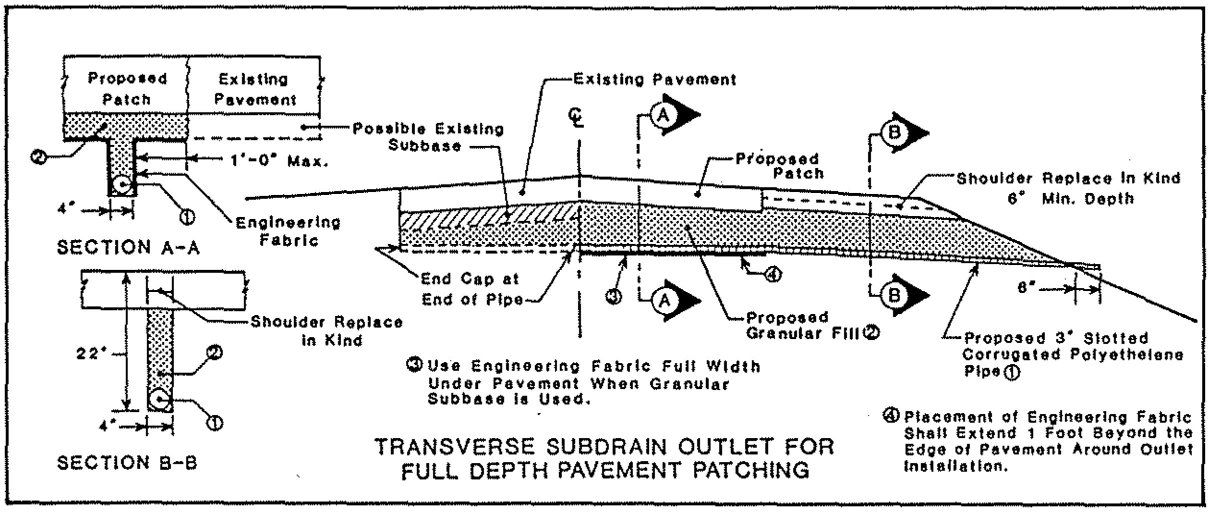
3. A joint resealing project should be performed over the entire project (trying to account for the long slab lengths).

4. A crack sealing program should be performed.

IA080288 - The original pavement was a 10-in [254 mm] JRCP with 76.5-ft [23.3 m] contraction joints and was constructed and opened to traffic in 1959. Full-depth repairs were placed in 1984, along with cement grout undersealing and diamond grinding. Only two full-depth repairs were surveyed. They contained 1.25-in [32 mm] diameter, 18-in [457 mm] long dowels (placed on 12-in [305 mm] centers), a fabric interlayer, transverse underdrains, and a drainable granular base. The joint was sawed and sealed after concrete placement. The repair design is shown in Figure 70.

The following table summarizes faulting of the surveyed joints [1 in = 25.4 mm]:

	REG CONT		REP APP		REP LV	
	outer	inner	outer	inner	outer	inner
Total Meeting Acceptance Criteria	100%	100%	100%	100%	100%	100%
Average Fault	0.04	0.02	0.07	0.07	0.07	0.07
ESALs Sustained By Joint Type (millions)	13.2	2.8	0.8	0.2	0.8	0.2



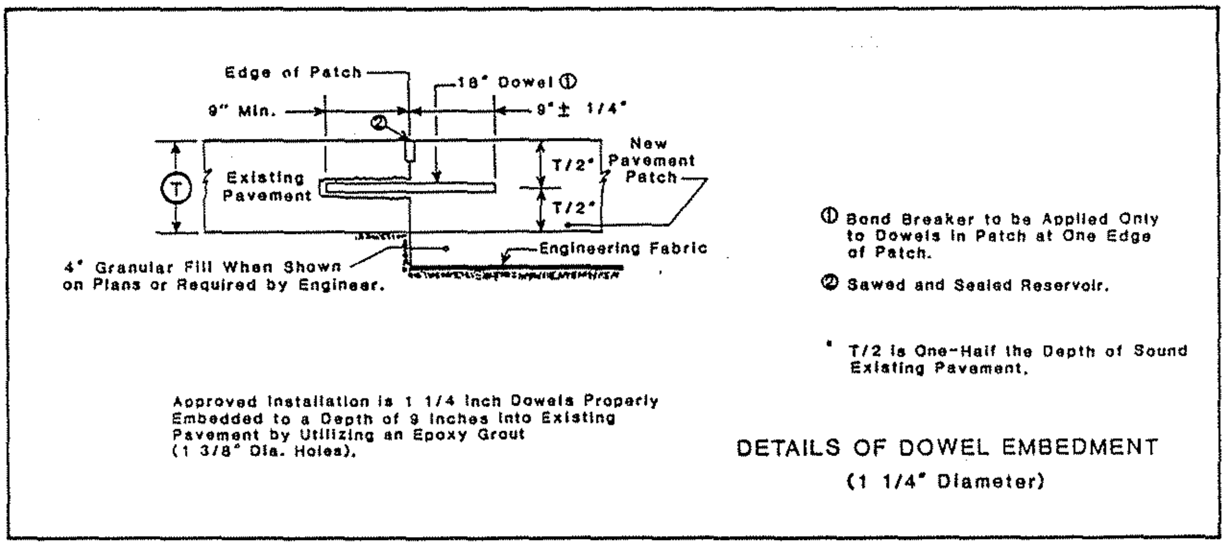


Figure 70. Illustration of Full-Depth Repair Design Used on IA080288 [1 in = 25.4 mm].

Faulting of the repairs is not a problem, as all measurements are well within acceptable limits. However, repair joint faulting is nearly double that of the original contraction joints after grinding. This particular project has historically exhibited a large degree of pumping.

The repairs themselves are uncracked, and the transverse repair joints are performing well. The joint sealant is still intact, keeping incompressibles out, and no spalling was observed. Joint widths ranged from 0.2 in to 1.4 in [5.1 mm to 35.6 mm]. The 1.4 in [35.6 mm] joint width was found on the leave side of one repair and is apparently an expansion joint.

The full-depth repairs successfully addressed the joint deterioration of the original pavement. They have withstood 1 year of traffic (nearly 1 million ESALs) without any signs of distress. The repairs are structurally sound and it is not believed they will deteriorate in the near future. The development of faulting will need to be monitored, however. Rapid development of faulting would be an indication that the epoxied dowels are not performing adequately.

Faulting of the undersealed repair leave joints in the first sample unit is much less than that of the non-undersealed repair leave joints in the second sample unit, which could indicate that the undersealing was beneficial. However, the larger faults could also be attributed to the apparent use of expansion joints in the repairs in the second sample unit.

The expected remaining life of this pavement before major rehabilitation is approximately 5 to 7 years from the date of survey (5 to 7 million ESALs). Thus major rehabilitation, in the form of additional full-depth repairs and/or diamond grinding, is expected to be needed as early as 1990 or as late as 1992.

IA035086 -The original pavement was a 10-in [229 mm] JRCP with 76.5-ft [23.3 m] contraction joints and was constructed and opened to traffic in 1965. This project features the evolution of the full-depth repair design used by the Iowa Department of Transportation. Full-depth repairs were placed from 1977 to present at approximately 70% of the joints to address spalling brought about by "D" cracking. Pressure relief joints were also placed within the project in 1980. The following table summarizes additions to the repair design (no additions have been added in the last two years).

Year	Addition
1980	Porous Material Beneath Repair
1981	Fabric Interlayer
1981	Transverse Drains
1983	Dowel Bars

The design of the 1985 full-depth repairs is the same as that shown in Figure 70. The dowels used were 1.25 in [32 mm] diameter, 18 in [457 mm] long, on 12 in [305 mm] centers. The following table summarizes the average full-depth repair joint faulting:

Summary of Observed Joint Faulting
(Faulting in inches [1 in = 25.4 mm]; no. of observations in parentheses)

Construction Year		79	81	83	84	85
Cum. ESAL (inner/outer)		0.6/3.0	0.4/2.2	0.3/1.2	0.1/0.6	0/0
INNER LANE	Approach	0.185 (2)	0.056 (7)	-	0.002 (13)	0.04 (23)
	Leave	0.135 (2)	0.071 (7)	-	0.051 (13)	0.09 (23)
OUTER LANE	Approach	0.268 (5)	0.250 (19	0.330 (1)	0.106 (17)	0.00 (1)
	Leave	0.070 (5)	0.233 (18	-0.04 (1)	-0.01 (17)	0.06 (1)
DOWELS		NO	NO	YES	YES	YES

This table illustrates that faulting is much lower on repairs placed after 1983, when dowel bars were incorporated in the design. The effect of traffic is also indicated by the significant differences in faulting between the inner and outer lanes. Curiously, these averages indicate that the approach joint is generally more faulted than the leave joints, which is contrary to what has been observed on other projects.

Most of the old repairs have assumed a tilted orientation (with respect to the original pavement), which is indicative of pumping. The positions of the newer repairs also indicates that some pumping may be occurring, but the magnitude of the faults suggest that the damage is minimal so far. A few of the newer repairs appear to be slightly raised or settled, which is attributed to repair overfilling or underfilling.

All of the full-depth repairs are structurally sound, but some are rough due to joint faulting and spalling. The repairs placed after 1983 are in the best overall condition, probably because these repairs included dowel bar load transfer systems and have been subjected to less traffic than the earlier repairs.

One of the 1981 outer lane repairs has cracked longitudinally near the center of the lane, but no other repair cracking was observed. Also, small corner breaks were found outside of a few of the older repairs, but the corners of the repairs themselves were never broken. As expected, the 1981 and 1979 repairs show the highest amount of spalling, particularly on the approach side of the repair. The older repairs (1981 and 1979) also exhibited poor sealant condition and incompressibles in the joints, which probably was the reason for the spalling. The 1984 and 1985 repairs exhibited very few significant spalls.

"D" cracking was not observed in the repairs themselves, but was found in the slabs adjacent to the repairs. This is probably due to incomplete removal of "D"-cracked concrete during repair work and subsequent progression of "D" cracking.

Medium-severity spalling was often found on either side of the approach joint of many of the pre-1983 repairs, which indicates the effectiveness of the dowels in reducing differential vertical joint movement. A summary of the spalling at repair locations is presented in Figure 71. The small corner breaks often accompanying many of the older repairs were also absent from the post-1983 repairs.

The full-depth repairs placed from 1979 through 1985 have addressed the joint spalling (due to "D" cracking) of the original pavement. However, faulting of many of the repairs remains a problem. Much of this roughness can be attributed to the older repairs, which have sustained more traffic and did not have all of the design improvements later incorporated. The lowest faulting seems to be at the 1984 repairs (only one 1983 repair was surveyed), which contain dowel bars.

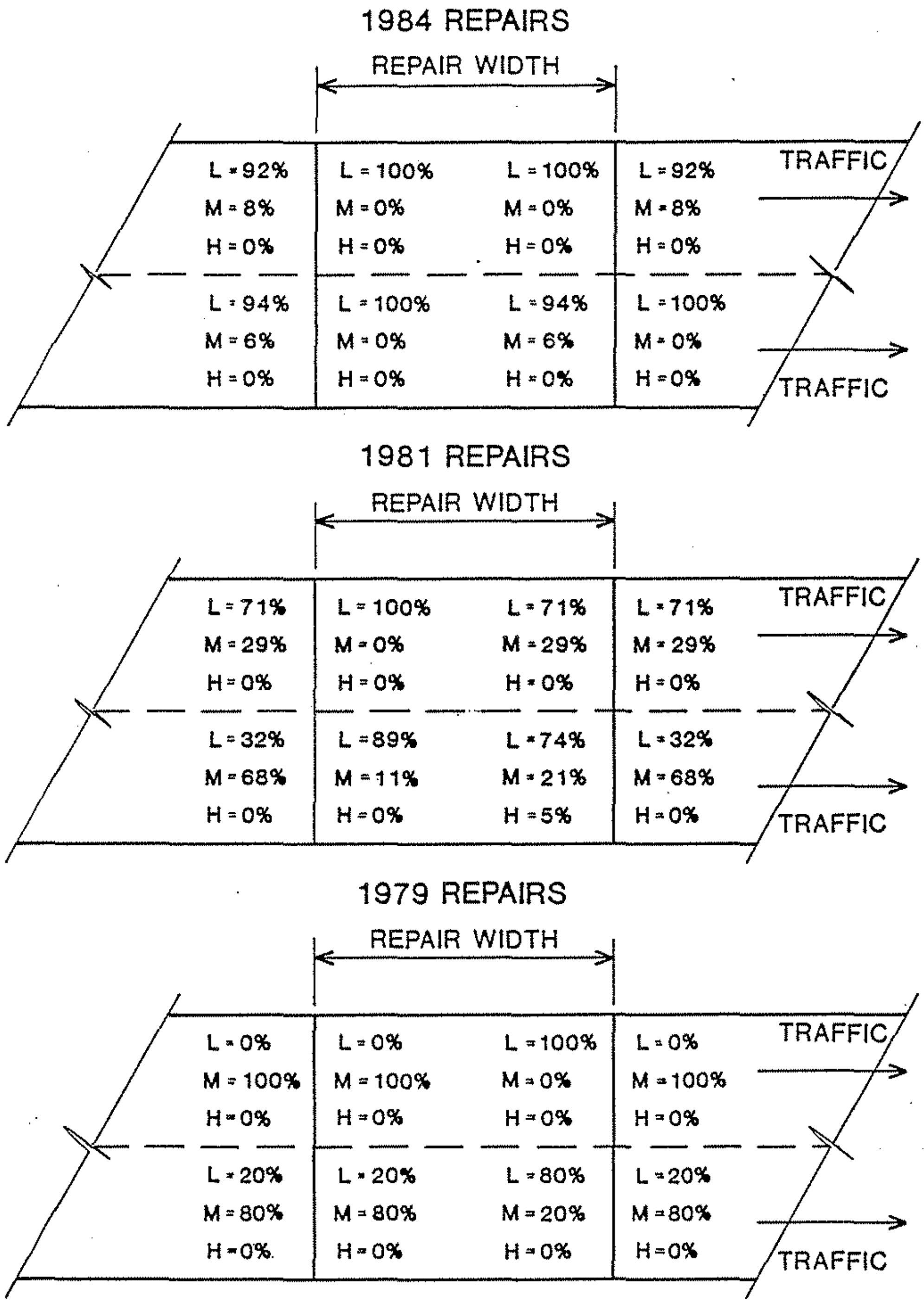
Of the four sets of full-depth repairs evaluated (1985, 1984, 1981, and 1979), the 1984 repairs are performing the best. The 1985 repairs are also performing well, but exhibit slightly higher levels of faulting than the 1984 repairs. This difference in performance may be due to differences in construction quality between the two years, since the repair designs are practically identical.

Overall, this rehabilitation project is considered successful from the standpoint that overall serviceability has been maintained. It is difficult to determine the effectiveness of design improvements such as the fabric interlayer, transverse drains, and the drainable base repair material because reductions in distress during these years are so small that they may be equally due to reductions in overall traffic carried by the repairs. The only design improvement that seems to have a significant effect was the inclusion of dowels beginning in the 1983 repairs.

Considering the current pavement condition and traffic levels, the expected life of this pavement is approximately 3 to 6 years from the date of survey (1.8 to 3.6 million ESALs). Thus major rehabilitation, in the form of additional repairs and diamond grinding or a structural overlay, is expected to be needed as early as 1988 or as late as 1991. The need for rehabilitation will probably be triggered by increasing deterioration of the pre-1983 repairs.

It is interesting to note the superior performance observed in the second sample unit. This was a short section of approximately 2 miles where an aggregate less susceptible to "D" cracking was used. The section is exhibiting very little distress and is performing well under the same traffic as the rest of the pavement, thus confirming that "D" cracking was a major cause of deterioration in the rest of the project.

LA010151 - The original pavement was a 10-in [254 mm] JRCP with 58.5-ft [17.8 m] contraction joints and was constructed and opened to traffic in 1971. Rehabilitation, in the form of full- and partial-depth repairs, cement grout undersealing, load transfer restoration, diamond grinding, joint resealing, crack repair, and longitudinal underdrains, was performed in 1984 for demonstration purposes only; the pavement was <u>not</u> in actual need of CPR work. Four full-depth repairs were placed over the 1800-ft [549 m] project in the outer lane only. The repairs were of length



NOTE: All undamaged joints and all joints with low severity spalling were rated as low severity.

Figure 71. Summary of Spalling at Repair Locations on IA035086.

6 ft [1.8 m] and used 1.125-in [29 mm] diameter, 18-in [457 mm] long dowels. They were laid out as follows:

Station	Dowel Spacing	Anchor Material
0+00	Unconventional	Yes
0+58	Conventional	Yes
1+17	Conventional	Yes
1+75	Conventional	No

Figure 72 gives the typical full-depth repair design, while Figure 73 illustrates the conventional and unconventional dowel spacings used. The repair joints were sawed and sealed with silicone after placement.

The following table summarizes faulting of the surveyed joints [1 in = 25.4 mm]:

	REG CONT		REP APP		REP LV	
	outer (ground)	inner	outer	inner	outer	inner
Total Meeting Acceptance Criteria	100%	100%	100%	N/A	100%	N/A
Average Fault	0.06	0.05	0.03	N/A	0.06	N/A
ESALs Sustained By Joint Type (millions)	10.0	2.0	1.4	N/A	1.4	N/A

Faulting measurements were all well within acceptable limits, but there is some difference in faulting of the repair joints. The following table compares the faulting of the observed full-depth repairs [1 in = 25.4 mm]:

Repair Design	Repair Approach Fault	Repair Leave Fault	Visible Pumping
Unconv-Epoxied	0.03	0.04	No
Conv-Epoxied	0.00	0.02	Yes-Low
Conv-Epoxied	0.01	0.07	Yes-High
Conv-Unepoxied	0.07	0.09	No

Faulting of repair joints where the dowels were epoxied is similar to that of the regular contraction joints that were ground in 1984. However, the faulting of the one repair without epoxied dowels was larger than the others. The full-depth repair with the "unconventional" dowel spacing and epoxied dowels exhibited the same magnitude of faulting as those with the conventional 12 in [305 mm] dowel spacing. It is interesting that the conventionally spaced, epoxied dowel repairs displayed visible signs of pumping, while the repair with the highest observed faults (conventional unepoxied) did not.

Most of the full-depth repair joints are performing well. Joint widths ranged from 0.6 in to 1.0 in [15 mm to 25 mm]. The sealant was still intact and no incompressibles were present in the joint. Where spalling was observed it was generally of low severity.

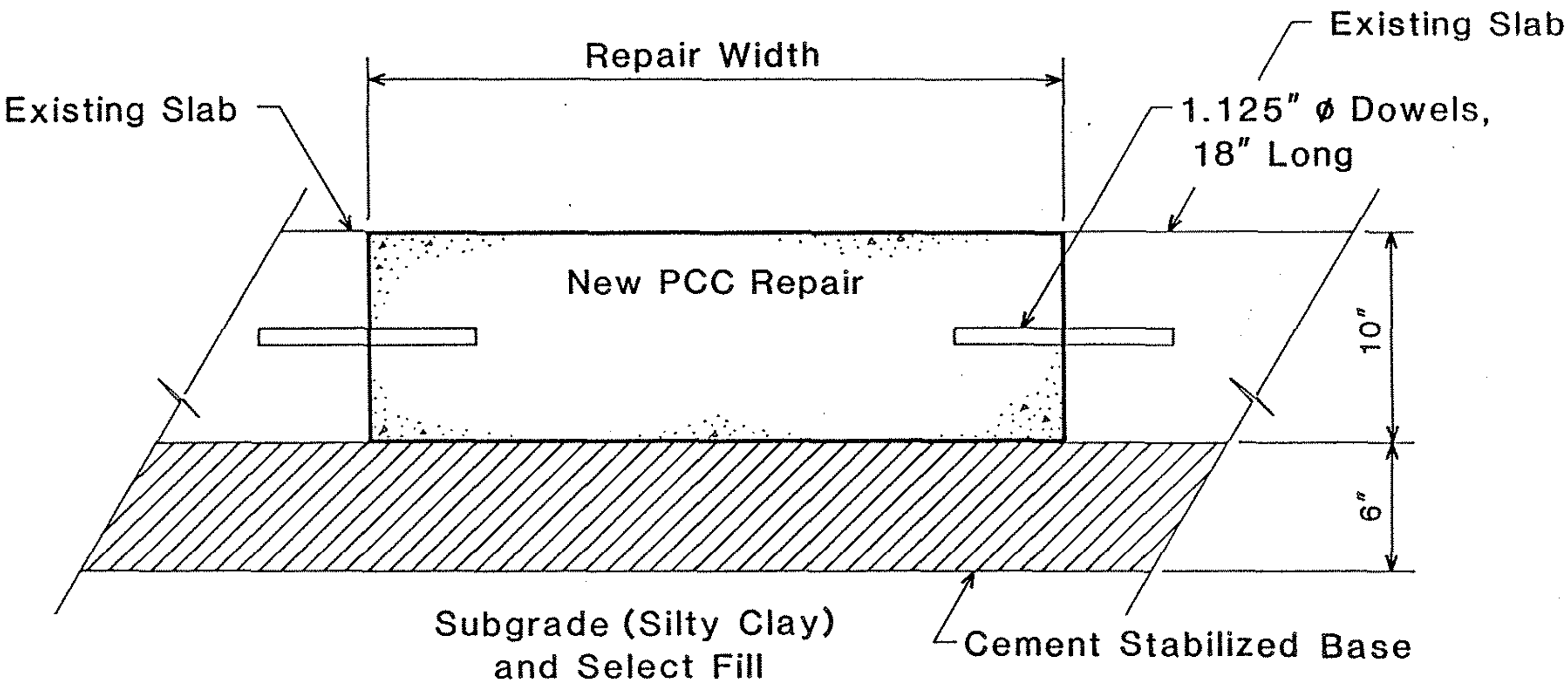
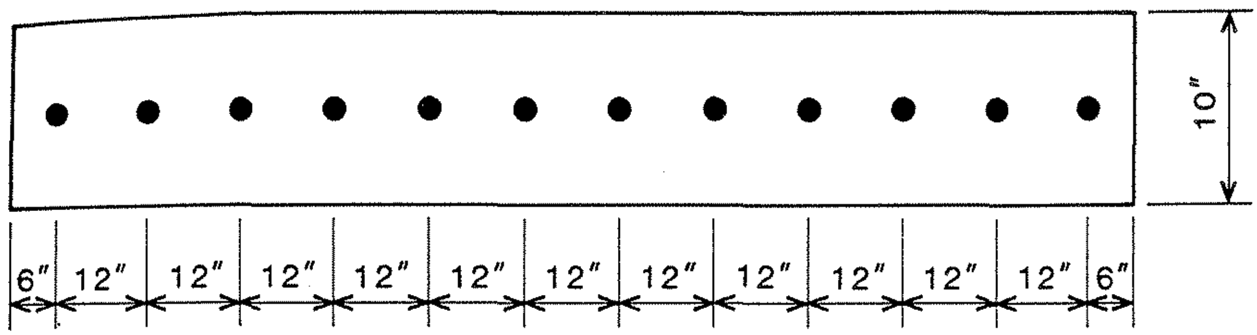
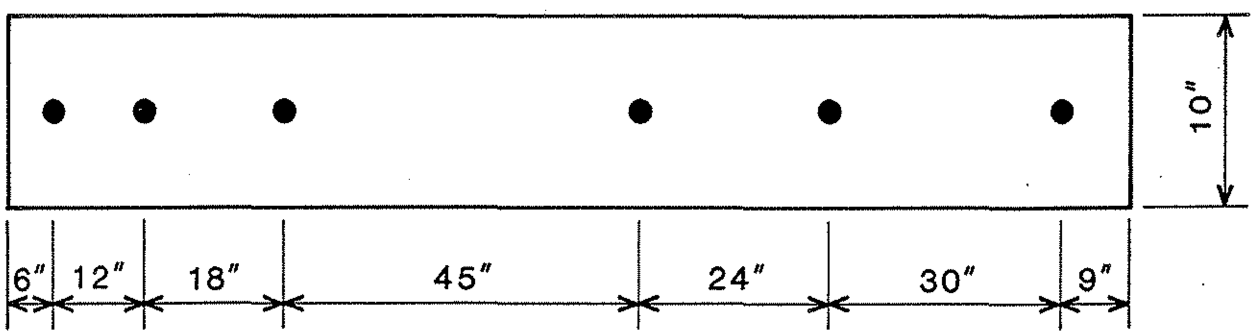


Figure 72. Illustration of Full-Depth Repair Design Used on LA010151
[1 in = 25.4 mm].



Conventional Dowel Bar Spacing



Unconventional Dowel Bar Spacing

Figure 73. Illustration of Dowel Spacing on LA010151 [1 in = 25.4 mm].

Since the full-depth repairs were placed as a demonstration for this CPR project, it is difficult to evaluate their success. Based on condition and performance data, the repairs do not appear to have performed as well as the original joints they replaced (the original joints exhibited no distresses). Nevertheless, since the repairs were placed as part of an experiment, the results obtained may outweigh reductions in performance.

Undersealing the full-depth repair areas would help stabilize the repairs, especially the ones exhibiting pumping and the one with the largest faulting (unconventionally spaced dowel bars).

This rehabilitation project is considered successful, due to the overall performance of each rehabilitation technique. Nearly all of the rehabilitation techniques are performing well after one year of service (1.4 million ESAL's), although the long-term performance of the rehabilitation will provide a better indication of their effectiveness.

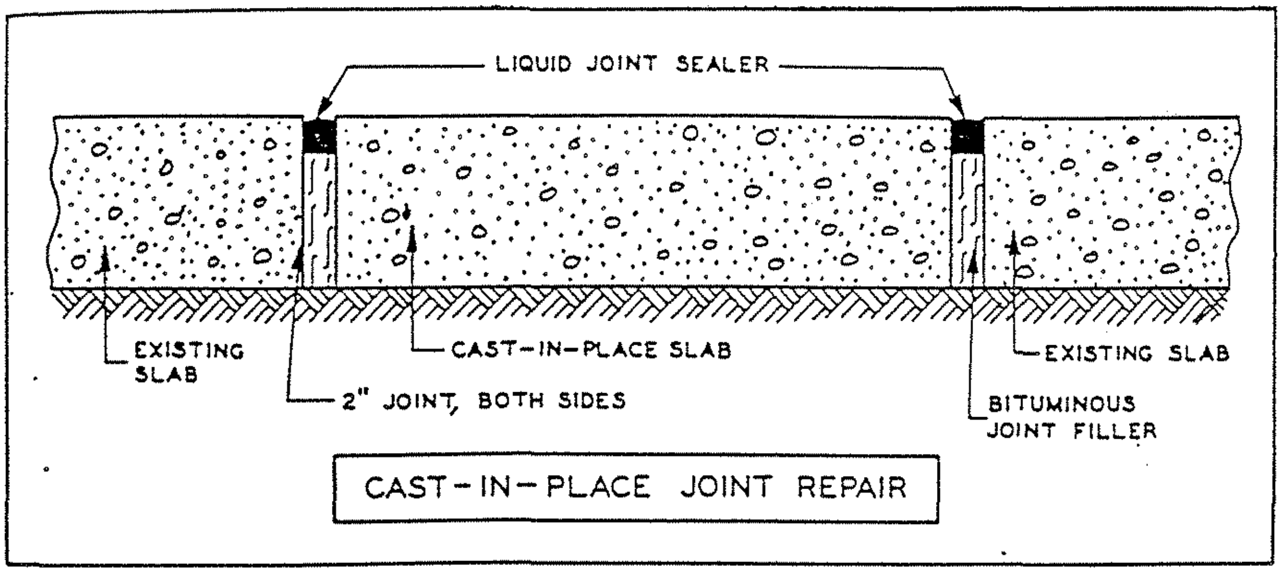
The expected life of this pavement is approximately six to eight years from the CPR (9 to 13 million ESALs). Thus major rehabilitation, in the form of additional repairs and/or an overlay, is not expected to be needed until 1990 to 1992.

MI127 - The original pavement was a 9-in [229 mm] JRCP with 99-ft [30-m] contraction joints and was constructed and opened to traffic in 1956. Pressure relief joints and full-depth repairs were installed in 1972 in the southbound lanes only. The northbound lanes served as a control section. Additional full-depth repairs were not placed in the southbound lanes again until 1979, whereas the northbound lanes had numerous full-depth repairs installed from 1972 to 1979. The full-depth repairs were both precast and cast in place (undowelled). Both designs used a 2-in [51-mm] bituminous joint filler on each side of the repair. Figure 74 illustrates the full-depth repair design used on this project.

The following table summarizes faulting of the surveyed joints [1 in = 25.4 mm]:

	SB REG CONT		NB REG CONT		SB REP APP		SB REP LV	
	outer	inner	outer	inner	outer	inner	outer	inner
Total Meeting Acceptance Criteria	97%	100%	100%	100%	33%	100%	0%	33%
Average Fault	0.05	0.04	0.11	0.07	0.20	0.13	0.51	0.24
ESALs Sustained By Joint Type (millions)	5.0	0.7	5.0	0.7	2.8	0.4	2.8	2.4

Only three full-depth repairs were surveyed: two cast-in-place repairs constructed in 1972 and one constructed in 1979. All of the surveyed repairs (and most of the other repairs observed) are structurally adequate but severely faulted, especially across the leave joint. The average



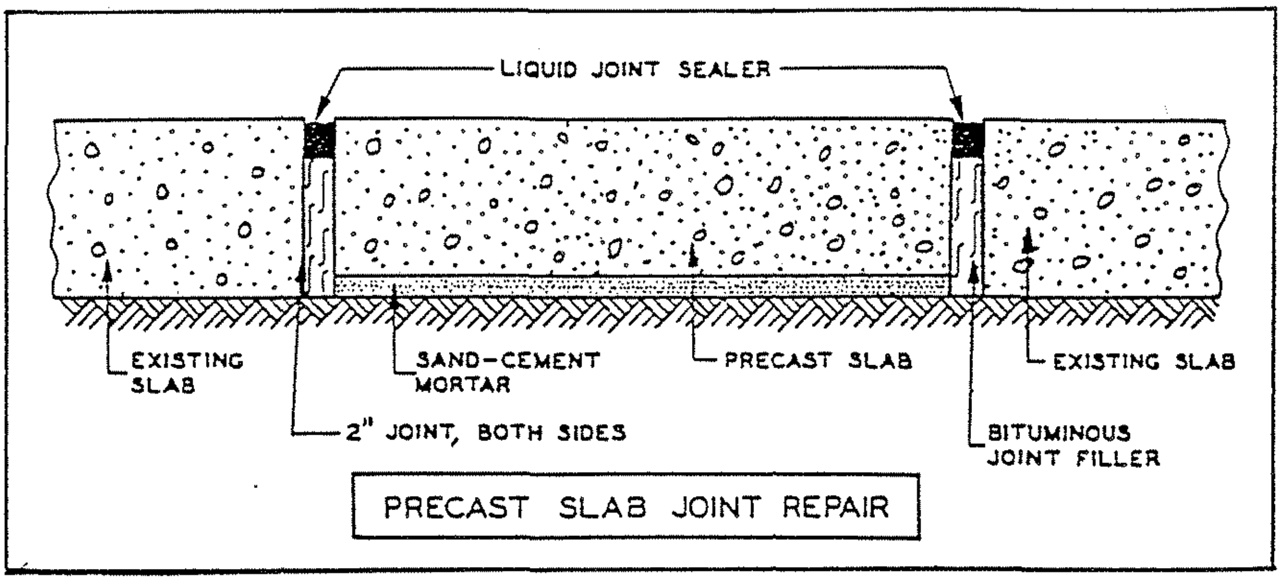


Figure 74. Illustration of Full-Depth Repair Designs Used on MI127 (115) [1 in = 25.4 mm].

measured repair faulting in the outer lane was 0.20 in [5.0 mm] for the approach joint and 0.51 in [13 mm] for the leave joint. Medium-severity pumping was also observed at the 1979 repair. No significant difference in repair performance (i.e., faulting, spalling and slab cracking) was detected between the 1972 and 1979 repairs.

The joint sealant of all three repairs was generally present, although incompressibles were often present. Only one surveyed repair joint exhibited significant spalling (medium severity). Most of the repair joints had closed from their original 2-in [51 mm] width to 1 in [25 mm] or less, and probably can accommodate some further expansion of the pavement.

The full-depth repairs were directed at joint spalling problems which were probably caused by compressive stresses in the pavement. Since only three full-depth repairs were surveyed (one constructed in 1979, two in 1972) and these were all cast-in-place, the performance of cast-in-place repairs cannot be compared to that of the precast repairs. Furthermore, the effects of time on repair performance cannot be analyzed for statistical significance with only three repairs. However, some observations can be made concerning the performance of the repairs themselves and their effect on the original pavement.

Excessive faulting of the repair joints remains the major problem of the repairs. It is apparent that a considerable amount water is entering the pavement through the deteriorated cracks, granular shoulder and unsealed transverse joints. There are no drains to remove this water and the fine-grained subgrade retains the moisture well enough to produce substantial pumping under relatively low traffic levels. The use of dowels or other mechanical load transfer devices together with the installation of drains to remove the moisture from the sand subbase would almost certainly have improved the performance of these repairs.

It is also likely that the use of repair expansion joints in the vicinity of the pressure relief joints contributed to excessive opening and deterioration of the transverse cracks in the adjacent slabs. Since compressive stress was probably relieved to some extent by the construction of the repairs, it is likely that the relief joints were not necessary.

Overuse of slab and repair expansion joints has caused excessive breakdown of the transverse cracks to the point where the rehabilitated lanes now require more rehabilitation than the control lanes. In fact, successful rehabilitation of the northbound (unrelieved) lanes would probably be accomplished with full-depth joint repairs, installation of subdrains and diamond grinding, while the southbound (relieved) lanes require extensive slab and joint repairs, a structural overlay or complete reconstruction.

Suggested improvements to the repair and relief joint designs include:

1. Provision of load transfer across all joints.

2. narrower expansion joints to prevent excessive movement of adjacent joints and cracks.

3. Greater separation between relief joints and concurrently placed full-depth repairs.

NE080189 - The original pavement was a 9-in [230 mm] JRCP with 46.5-ft [14.2 m] contraction joints, constructed and opened to traffic in 1965. Full-depth repairs without mechanical load transfer devices were placed in 1981 at about 27% of the driving lane joints and 8% of the passing lane joints to address spalls caused by the presence of reactive aggregate. Some major cracks were also repaired. Additional repairs were placed in 1985 to address further joint deterioration, but were not included in the surveyed sample units. Pressure relief joints were also placed on the project. Figure 75 gives the design of the 1981 full-depth repairs.

The following table summarizes faulting of the surveyed joints [1 in = 25.4 mm]:

	REG CONT outer	REG CONT inner	REP APP outer	REP APP inner	REP LV outer	REP LV inner
Total Meeting Acceptance Criteria	100%	100%	100%	100%	73%	83%
Average Fault	0.02	0.01	0.05	0.03	0.15	0.10
ESALs Sustained By Joint Type (millions)	12.0	2.5	3.0	0.7	3.0	0.7

Faulting of the repairs is within acceptable limits, except for one in four of the repair leave joints. This table shows that the original contraction joints have not faulted significantly under heavy traffic, probably due to the excellent subgrade on which the highway was constructed. The repair joints are displaying much higher faults (despite having carried less traffic), which may be attributed to the lack of dowels in the design. Several repair joint faults exceed 0.20 in [5.1 mm]. However, some of this faulting can probably be attributed to overfilling or underfilling the repair). The data collected indicates a tendency toward tilted and settled repairs.

In general, the full-depth repairs placed in 1982 are performing well. Spalling is generally not a problem, but when it exists it is usually on the original pavement side of the approach joint and of low severity. Reactive aggregate distress was observed near the repair joints and was the primary cause for any spalling located on the original pavement side of the repairs. Occasionally, some of the joint sealant was also extruded, probably due to poor sealing techniques.

Thirty percent of the repairs exhibited low-severity longitudinal slab cracking (see Figure 76). This may be due to either a mechanism similar to the edge punchout in CRCP, or to placement of the repairs in the morning, after which the heat of the afternoon causes the original pavement slabs to expand and crush the new repair. Longer repairs lengths, coupled with late afternoon placement, might eliminate this problem.

One repair also exhibited transverse cracking, but this occurred on a long repair. The use of reinforcement may have prevented this cracking.

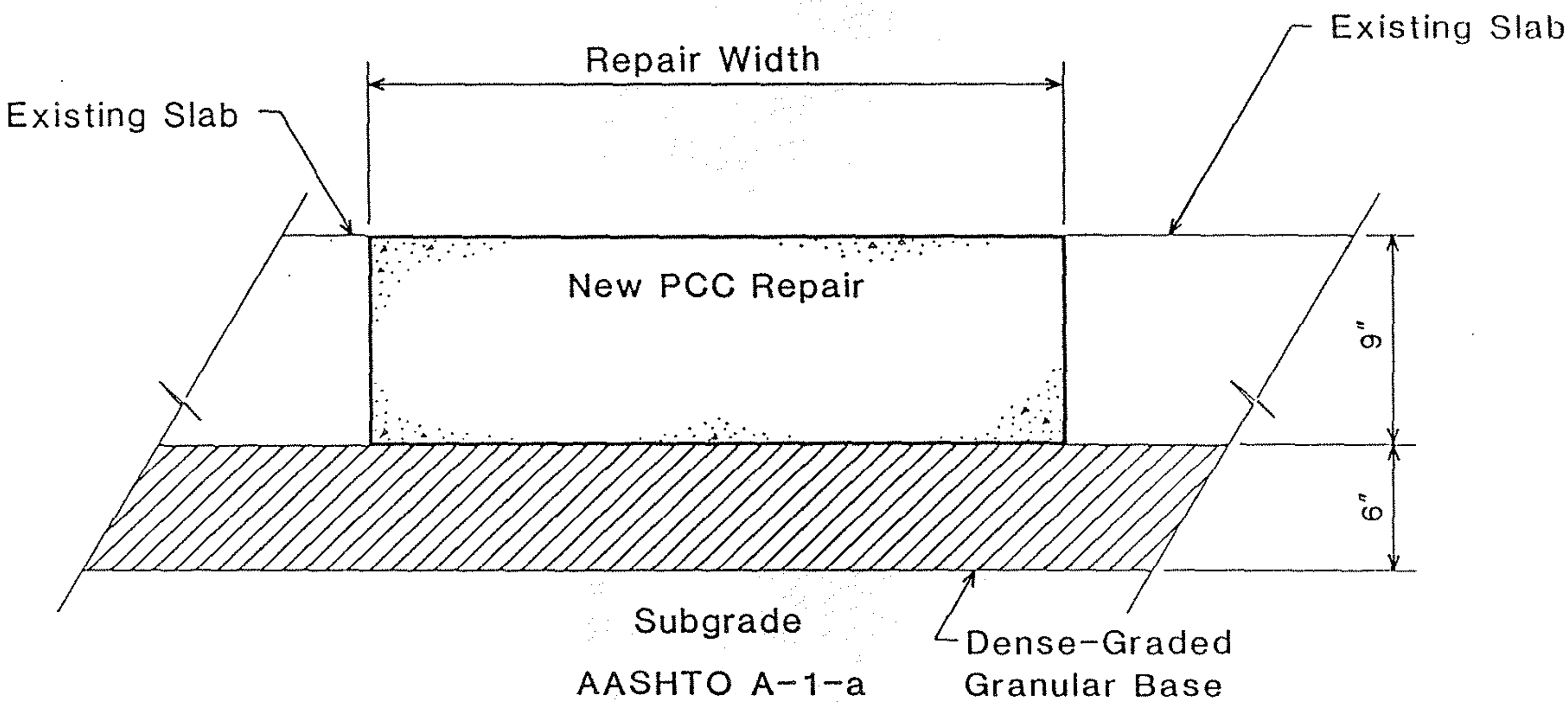


Figure 75. Illustration of Full-Depth Repair Design Used on NE080189
[1 in = 25.4 mm].

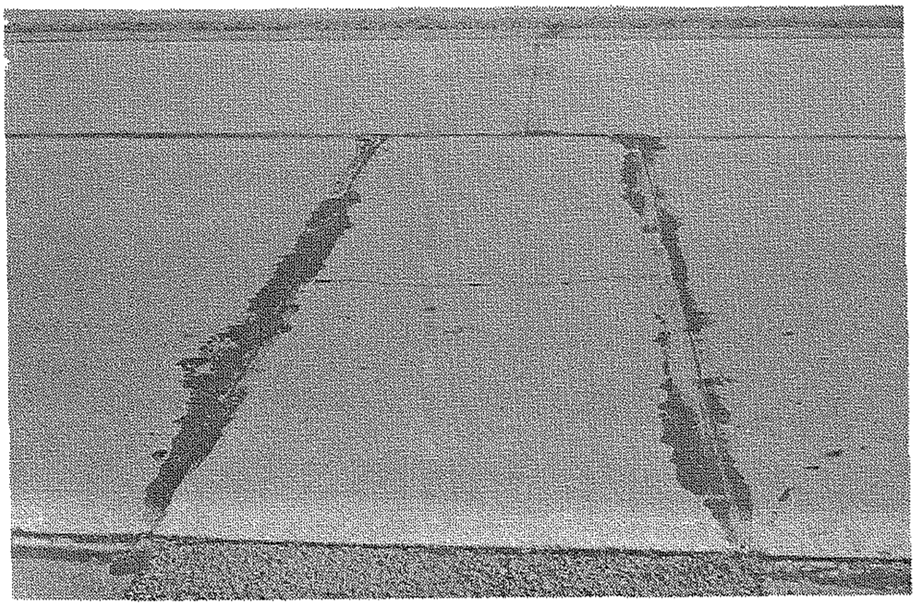


Figure 76. Photo of Typical Full-Depth Repair on NE080189.

The full-depth repairs placed in 1981 were directed at joint spalling of the original pavement due to the presence of reactive aggregate. This repair technique successfully addressed the spalling problems, as few spalls are present near the repair areas. The few spalls that were observed were generally in the original pavement and are probably due to expansion of the concrete due to the alkali-aggregate reaction. Correction of the spalling probably significantly reduced the overall roughness of the pavement. Faulting of the pavement remains a problem, however. An effective mechanical load transfer system might have reduced faulting and settlement of repairs that occurred during periods of slab contraction. Because of the continued deterioration of the unrepaired joints (due to reactive aggregate), additional repairs were scheduled for July, 1985.

Overall, this rehabilitation project is considered successful due to the performance of the full-depth repairs. Most of the joints in both lanes are smooth, and spalling is not a concern.

The expected life of this pavement is approximately six to nine years from the time of installation of the full-depth repairs (6 to 9 million ESALs). Major rehabilitation, in the form of additional repairs and/or an overlay, is expected to be needed as early as 1987 or as late as 1990.

The major problem on this project is the presence of reactive aggregate, which is the primary cause of the joint spalling. There is little that can be done now to deal with this problem directly, other than installing full- and partial-depth repairs when needed.

NE080210 - The original pavement was a 9-in [230 mm] JRCP with 46.5-ft [14.2 m] contraction joints and was constructed and opened to traffic in 1964. Full-depth repairs without mechanical load transfer devices were placed in 1979 at about 86% of the driving lane joints and 88% of the passing lane joints to address spalls caused by the presence of reactive aggregate. A few repairs were also placed at major cracks. Pressure relief joints were also placed on the project. Figure 75 gives the typical full-depth repair design on this project.

The following table summarizes faulting of the surveyed joints [1 in = 25.4 mm]:

	REG CONT		REP APP		REP LV	
	outer	inner	outer	inner	outer	inner
Total Meeting Acceptance Criteria	100%	100%	98%	99%	81%	90%
Average Fault	0.03	0.02	0.06	0.13	0.12	0.08
ESALs Sustained By Joint Type (millions)	14.0	1.0	6.2	0.5	6.2	0.5

The original contraction joints have not faulted significantly under the heavy traffic, probably due to the excellent subgrade present. The repair joints are displaying much higher faults (despite having carried less traffic), which may be attributed to the lack of dowels in the design. Faulting measurements are generally within acceptable limits, although typical repair joints were 4 to 6 times rougher than the original contraction joints. Most of the repairs are either raised (constructed high) or are tilted (as one would expect if pumping was taking place) relative to the rest of the pavement. Pumping was observed as a stain on the shoulder near several of the full-depth repairs.

Nearly half of the repairs surveyed exhibited low- and medium-severity longitudinal cracking (see Figure 77). These cracks nearly always occurred at the center of the lane and may be caused either by a mechanism similar to the development of edge punchouts in CRCP, or placement of the repairs in the morning such that the heat of the afternoon caused the original concrete slabs to expand and crush the newly placed repair. Longer repair lengths, coupled with late afternoon placement, may eliminate this problem.

Reactive aggregate distress was observed in the original pavement near the repair joints and was the primary cause for any spalling on the original pavement side of the repairs. It was not found within the repairs themselves.

Over half of the transverse contraction joint associated with the full-depth repairs are performing adequately. Joint widths averaged 0.21 in [5.3 mm]. In some cases, the joint sealant had extruded, probably due to poor sealing techniques.

The full-depth repairs placed in 1979 were directed at joint spalling of the original pavement due to the presence of reactive aggregate. This repair technique successfully addressed the spalling problems at the joint, but subsequently new spalls have developed on the approach and leave sides of the repairs. The repairs themselves have little, if any, spalling.

The use of dowel bars in the design and proper sealing of the transverse repair joints might have improved the performance of the repairs. The use of dowels alone might have held faulting of the repair joints to a level comparable to that of the original contraction joints.

Overall, this rehabilitation project is considered successful, due to the satisfactory performance of the full-depth repairs. Most of the joints in both lanes are generally smooth, and spalling is only a problem on the approach and leave slabs outside of the repairs.

The expected life of this pavement is approximately seven to nine years from the time of installation of the full-depth repairs (4.0 to 5.6 million ESALs). Major rehabilitation, in the form of additional repairs and/or an overlay, is expected to be needed as early as 1986 or as late as 1988.

The major problem with this pavement is the expansion of reactive aggregate, which is the primary cause for the joint spalling. There is little that can be done now to deal with this problem directly, other than full-depth repairs when needed.

Figure 77. Photo of Full-Depth Repair and Reactive Aggregate Distress on NE080210.

NE080256 - The original pavement was a 9-in [230 mm] JRCP with 46.5-ft [14.2 m] contraction joints and was constructed and opened to traffic in 1963. Full-depth repairs without mechanical load transfer devices were placed in 1980 at about 36% of the driving lane joints and 15% of the passing lane joints to address spalls caused by the presence of reactive aggregate. Some major cracks were also repaired. Pressure relief joints were also installed within the project. The joints were formed with a trowel and sealed. Figure 75 illustrates the typical full-depth repair design for this project.

The following table summarizes faulting of the surveyed joints [1 in = 25.4 mm]:

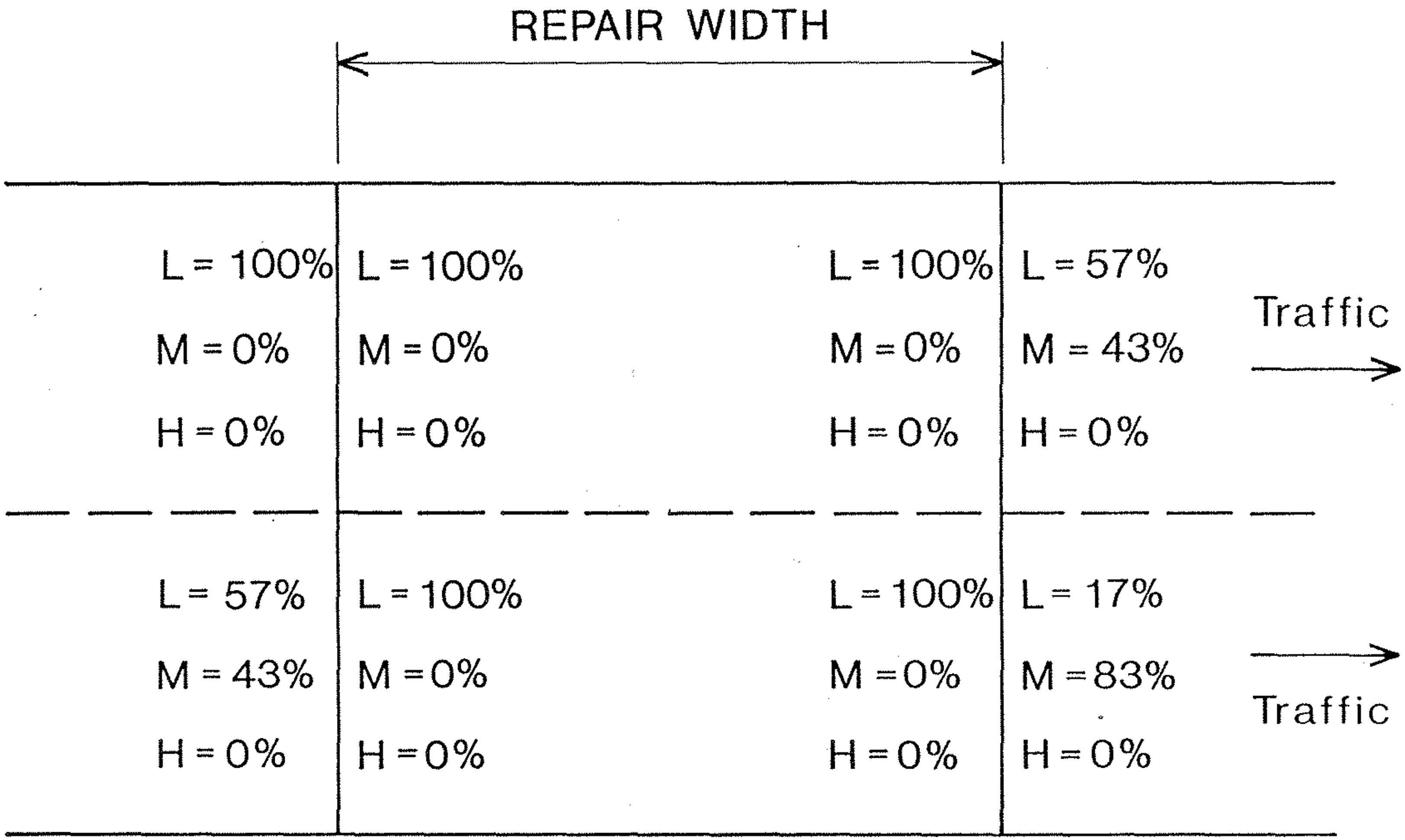
	REG CONT		REP APP		REP LV	
	outer	inner	outer	inner	outer	inner
Total Meeting Acceptance Criteria	93%	100%	93%	71%	92%	100%
Average Fault	0.04	0.04	0.07	0.13	0.11	0.06
ESALs Sustained By Joint Type (millions)	16.0	1.2	5.6	0.4	5.6	0.4

Repair joint faulting is generally within acceptable limits, although it is much higher than original contraction joint faulting. The original contraction joints have not faulted significantly under heavy traffic, probably due to the excellent subgrade on which the highway was constructed. The repair joints are displaying much higher faults, despite having carried less traffic, which may be attributed to the lack of dowels in the design. Pumping was observed as a stain on the shoulder near several of the full-depth repairs.

Most of the transverse contraction joints associated with the full-depth repairs are performing adequately. Joint widths averaged about 0.22 in [5.6 mm]. Occasionally, the joint sealant had extruded, probably due to poor sealing techniques. When spalling was observed, it was generally outside of the repair. This was due to the reactive aggregate present in the original concrete pavement. Little spalling was located within the repair itself. Figure 78 gives the summary of spalling at the repair locations. Several of the surveyed slabs display low- and medium-severity longitudinal cracking.

The full-depth repairs placed in 1980 were directed at joint spalling of the original pavement due to expansion of reactive aggregate. This repair technique successfully addressed the spalling at the joints, but subsequently new spalls have developed on the approach and leave sides of the repairs. The repairs themselves exhibit little, if any, spalling.

The repairs also provide a relatively smooth riding surface and the majority of the repair joint faults fall in the "acceptable range" of -0.20 in to 0.20 in [-5.1 mm to 5.1 mm]. However, the repair joints have generally faulted more after five years of service than the original



NOTE: All undamaged joints and all joints with low severity spalling were rated as low severity.

Figure 78. Summary of Spalling at Repair Location on NE080256.

contraction joints have since the pavement was constructed. Partial-depth repairs might have addressed the spalling without significantly affecting the roughness of the pavement, but might not have been as cost-effective.

The performance of the transverse repair joints might have been improved through the use of mechanical joint load transfer devices and good joint sealing practices. Load transfer is especially important because the aggregate interlock relied on in this design is probably not completely effective until the pavement has expanded (alkali reaction) sufficiently to insure constant aggregate interlock.

Overall, this rehabilitation project is considered successful, due to the good performance of the full-depth repairs and absence of much recent pressure damage. Most of the joints in both lanes are smooth, and spalling is only a problem on the approach and leave slabs outside of the repairs.

The expected life of this pavement is approximately six to nine years since the installation of the full-depth repairs (8.4 to 12.6 million ESALs). Major rehabilitation, in the form of additional repairs and/or an overlay, is expected to be needed as early as 1986 or as late as 1989.

The major problem of the pavement is the expansion of reactive aggregate, which is the primary cause for the joint spalling. There is little that can be done now to deal with this problem directly, other than full- and partial-depth repairs when needed.

NE080279- The original pavement was a 9-in [230 mm] JRCP with 46.5-ft [14.2 m] contraction joints and was constructed and opened to traffic in 1963. Full-depth repairs without mechanical load transfer devices were placed in 1982 at major cracks and about 27% of the driving lane joints and 8% of the passing lane joints to address spalls caused by the presence of reactive aggregate problems. The only full-depth repairs surveyed were those located at major cracks. Figure 75 illustrates the typical full-depth repair design for this project. Transverse joints were formed with a trowel. Other rehabilitation techniques performed on this section include partial-depth repairs, pressure relief joints, and joint resealing.

The following table summarizes faulting of the surveyed joints [1 in = 25.4 mm]:

	REG CONT		REP APP		REP LV	
	outer	inner	outer	inner	outer	inner
Total Meeting Acceptance Criteria	100%	100%	100%	100%	100%	100%
Average Fault	0.05	0.01	0.07	0.04	0.10	0.10
ESALs Sustained By Joint Type (millions)	14.0	3.2	3.9	0.8	3.9	0.8

The original contraction joints have not faulted significantly under heavy traffic, probably due to the excellent subgrade present. The repair joints are displaying much higher faults (despite having carried less traffic), which may be attributed to the lack of dowels in the design. The repair leave joints consistently display higher faults than the repair approach joints.

The full-depth repair joints are performing well. Joint widths averaged about 0.36 in [9.1 mm]. Repair leave joints were generally wider and more severely faulted than the approach joints. Some of the joint sealant had extruded, probably due to poor sealing techniques.

Repairs located in areas of highly deteriorated pavement were also somewhat deteriorated and exhibited longitudinal cracking near the center of the lane. This may be due to the placement of the repairs in the morning, in that the heat of the afternoon may cause sufficient expansion of the existing concrete slabs to crush the newly placed repairs.

Although reactive aggregate distress was present on the original pavement adjacent to the repairs, it was not found within the repairs themselves. Low-severity spalling due to reactive aggregate distress was observed on the adjacent slab on either side of several repairs, but not within the repairs themselves.

The full-depth repairs placed in 1982 were directed at joint and crack spalling caused by reactive aggregate distress. The repairs have successfully addressed this problem and have generally provided a smooth, serviceable pavement surface. Joint faulting was minimal in spite of the lack of mechanical load transfer devices. This is probably attributed to the lack of free moisture found in this region (due to high subgrade permeability and low annual precipitation), low subgrade/base erodibility, and good deflection load transfer since the expansive characteristics of the pavement increase the effectiveness of the aggregate interlock.

Some of the repairs have cracked longitudinally near the center of the lane and are located in areas of more severe pavement deterioration. It is not known whether the repair boundary selection failed to include all of the deteriorated pavement or if the adjacent pavement deteriorated after the repair was placed. Careful consideration to repair boundaries should be given to insure that all major structural distresses are included.

Joint faulting might have been reduced with mechanical load transfer devices. Adjacent slab spalling might have been reduced by sawing and sealing the transverse joints immediately after placement of the repairs.

Overall, this rehabilitation project is considered successful, due to the good performance of the full-depth repairs and joint resealing. Most of the joints in both lanes are smooth and well sealed.

The repairs are expected to provide another two to five years of good performance before adjacent slab spalling (due to the reactive aggregate) becomes unacceptable. Thus major rehabilitation, in the form of additional repairs and/or an overlay, is expected to be needed as early as 1988 or as late as 1991. Some areas of the original pavement are badly cracked and require rehabilitation now.

Reactive aggregate expansion, which has been so destructive to other projects in Nebraska (i.e., NE080189, NE080210, and NE080256), has not been a major problem on this project. Although present, it has not resulted in major spalling problems near the joints. This is attributed to this project utilizing limestone as its coarse aggregate, whereas the other Nebraska projects suffering from reactive aggregate distress used North Platte River gravel, which has been linked to reactive aggregate distress.

NE080382 - The original pavement was a 9-in [230 mm] JRCP with 46.5-ft [14.2 m] contraction joints and was constructed and opened to traffic in 1962. Full-depth repairs (no dowels) were placed in 1982 at about 13% of the driving lane joints and 9% of the passing lane joints to address spalls caused reactive aggregate. A few repairs were also placed at major cracks. Figure 79 gives the design of the full-depth repairs on this project. Other rehabilitation techniques applied include partial-depth repairs, pressure relief joints, and joint resealing.

The following table summarizes faulting of the surveyed joints [1 in = 25.4 mm]:

	REG CONT		REP APP		REP LV	
	outer	inner	outer	inner	outer	inner
Total Meeting Acceptance Criteria	74%	100%	100%	100%	13%	20%
Average Fault	0.14	0.02	0.02	0.04	0.40	0.42
ESALs Sustained By Joint Type (millions)	17.0	5.0	3.5	1.3	3.5	1.3

Faulting was a particular problem for the repair leave joints. Less than 20% of the repair leave joints met the acceptance criteria, while all of the approach joints were acceptable. Most of the repair joints have faulted in a way that indicates movement of the supporting materials from the leave side to the approach side of both the approach and leave repair joints. The magnitude of the faulting is probably due to the fine-grained subgrade present. This has resulted in much higher faulting (for both repair joints and regular contraction joints) than has been experienced by other Nebraska projects which were constructed on a better subgrade.

Several of the inner lane repairs exhibited longitudinal cracking. This has been attributed to the repairs being left open for an extended time (3-4 days) during a wet period, which permitted water to soften the subgrade prior to placement of the repair. The result was that the patches rocked under traffic and deteriorated quickly.

Repair joint performance is mixed. Repair approach joints were generally in better condition than leave joints, exhibiting narrower widths, less faulting and less spalling. Where spalling was observed, it was generally located on the adjacent slab, although the repair itself was spalled in some instances (see Figure 80). Some of this distress can be attributed to the reactive aggregate in the mainline pavement.

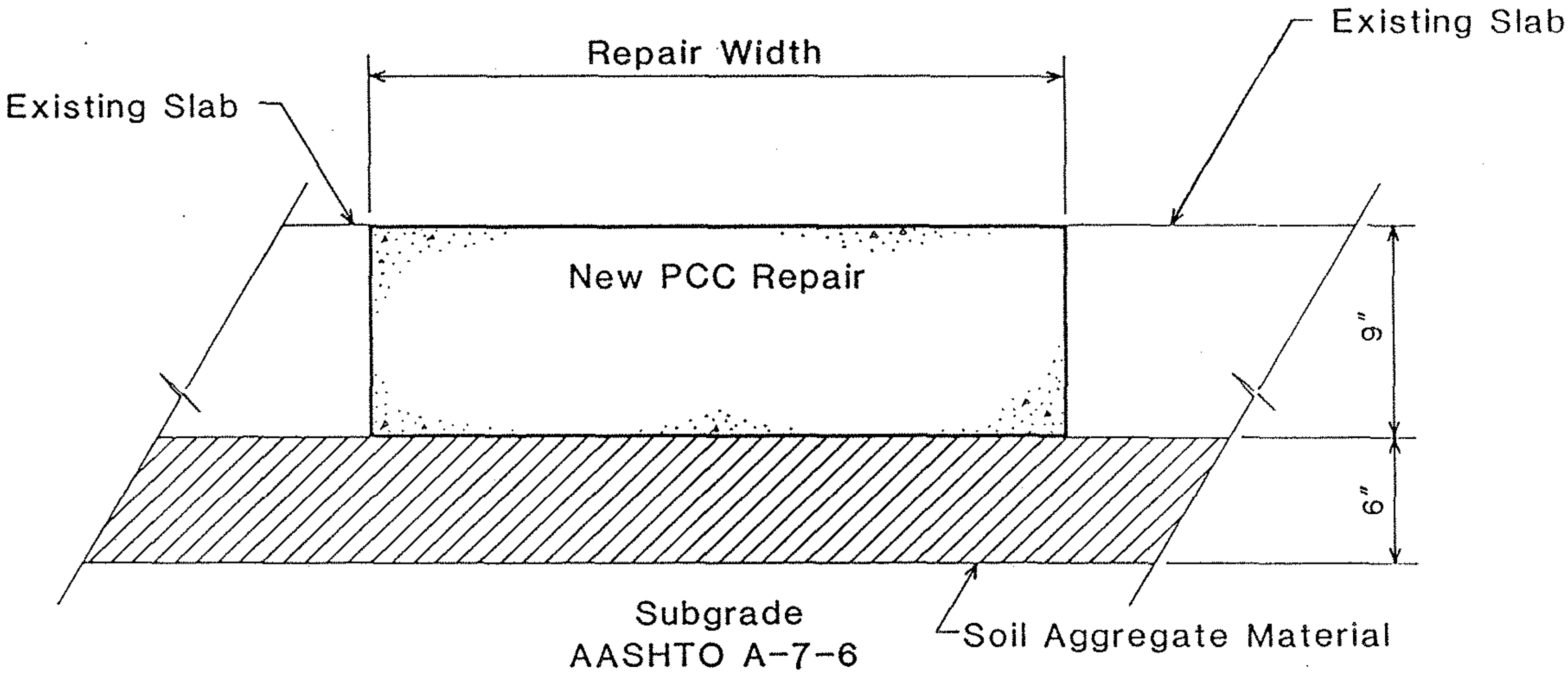
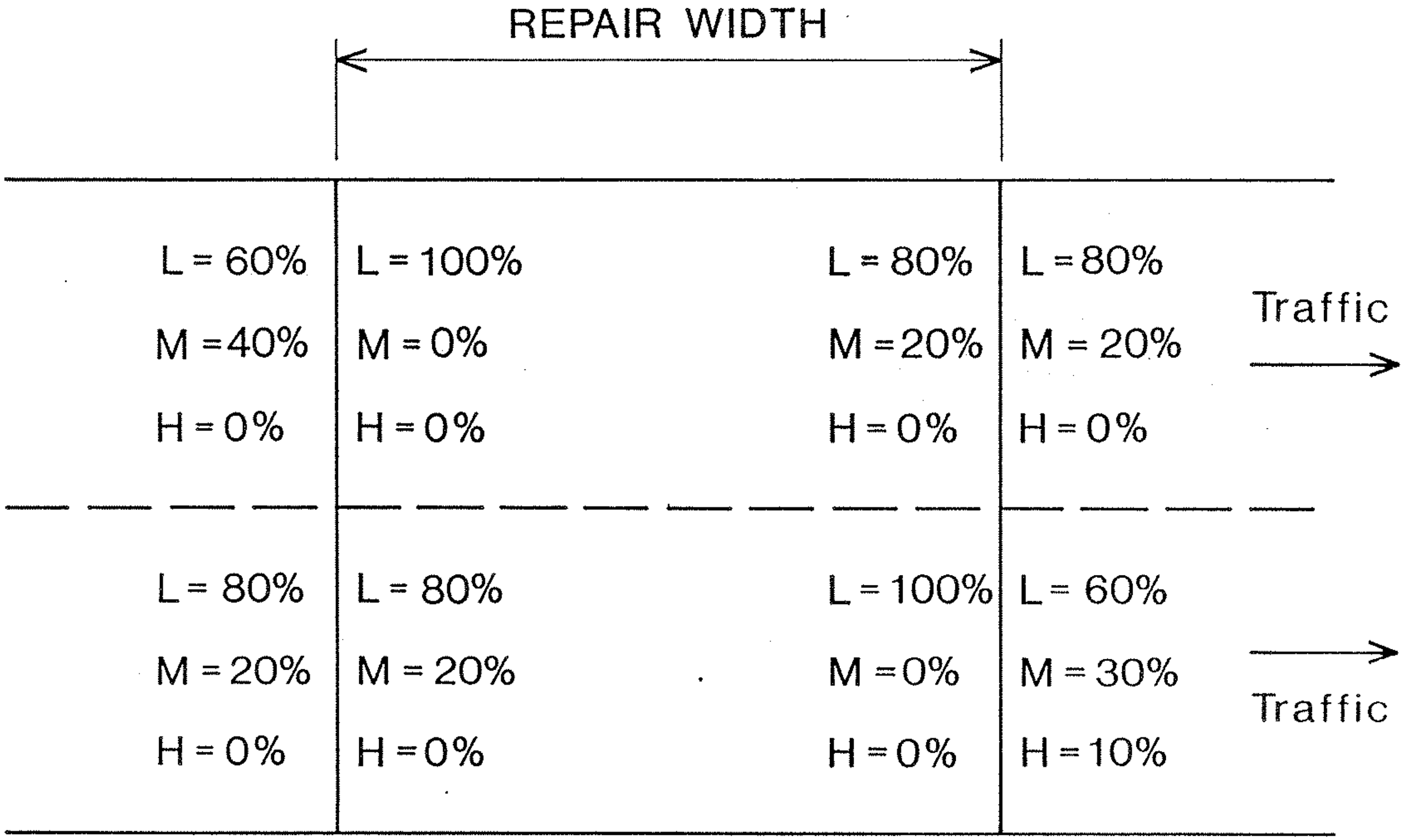


Figure 79. Illustration of Full-Depth Repair Design Used on NE080382
[1 in = 25.4 mm].



NOTE: All undamaged joints and all joints with low severity spalling were rated as low severity.

Figure 80. Summary of Spalling at Repair Location on NE080382.

The full-depth repairs placed in 1982 were directed at joint and crack spalling of the original pavement due to reactive aggregate. The repair design that was used depended heavily on the expansive nature of the pavement to insure aggregate interlock load transfer. Apparently this was not achieved, either because the pavement was not as expansive as had been originally assumed or because the relief provided by the relief joints and the full-depth repair installation sufficiently accommodated the expansive stress in the pavement. This problem was compounded by high traffic volumes, with the resulting joint movements producing large faulting.

The repair performance may have been increased by:

1. Better transverse and longitudinal joint sealing practices (including sawing and sealing the repair joints as soon as possible) to prevent water from entering the pavement system during the periods when excess moisture is present.

2. Undersealing after placement of the repairs to make the supporting material less erodible.

3. The use of dowels in the repairs to reduce deflections.

Overall, this rehabilitation project is considered moderately successful because the full- and partial-depth repairs have reduced the roughness caused by reactive aggregate spalling at the pavement joints, although the full-depth repairs have become somewhat rough themselves.

While the original pavement has several years of serviceable life remaining, the full-depth repairs placed in 1982 should be replaced immediately. The joints will require resealing within the next few years and a longer-life sealant is recommended.

NE080404 - The original pavement was a 10-in [254 mm] JPCP with 16.3-ft [5.0 m] contraction joints and was constructed and opened to traffic in 1960. Full-depth repairs without load transfer devices were placed in 1984. Approximately 3% of the joints were replaced. Figure 81 gives the full-depth repair design used on this project. Partial-depth repairs and joint resealing was also performed on the project.

The following table summarizes faulting of the surveyed joints [1 in = 25.4 mm]:

	REG CONT		REP APP		REP LV	
	outer	inner	outer	inner	outer	inner
Total Meeting Acceptance Criteria	70%	94%	60%	100%	40%	20%
Average Fault	0.17	0.06	0.09	0.08	0.20	0.11
ESALs Sustained By Joint Type (millions)	16.0	4.2	1.2	0.4	1.2	0.4

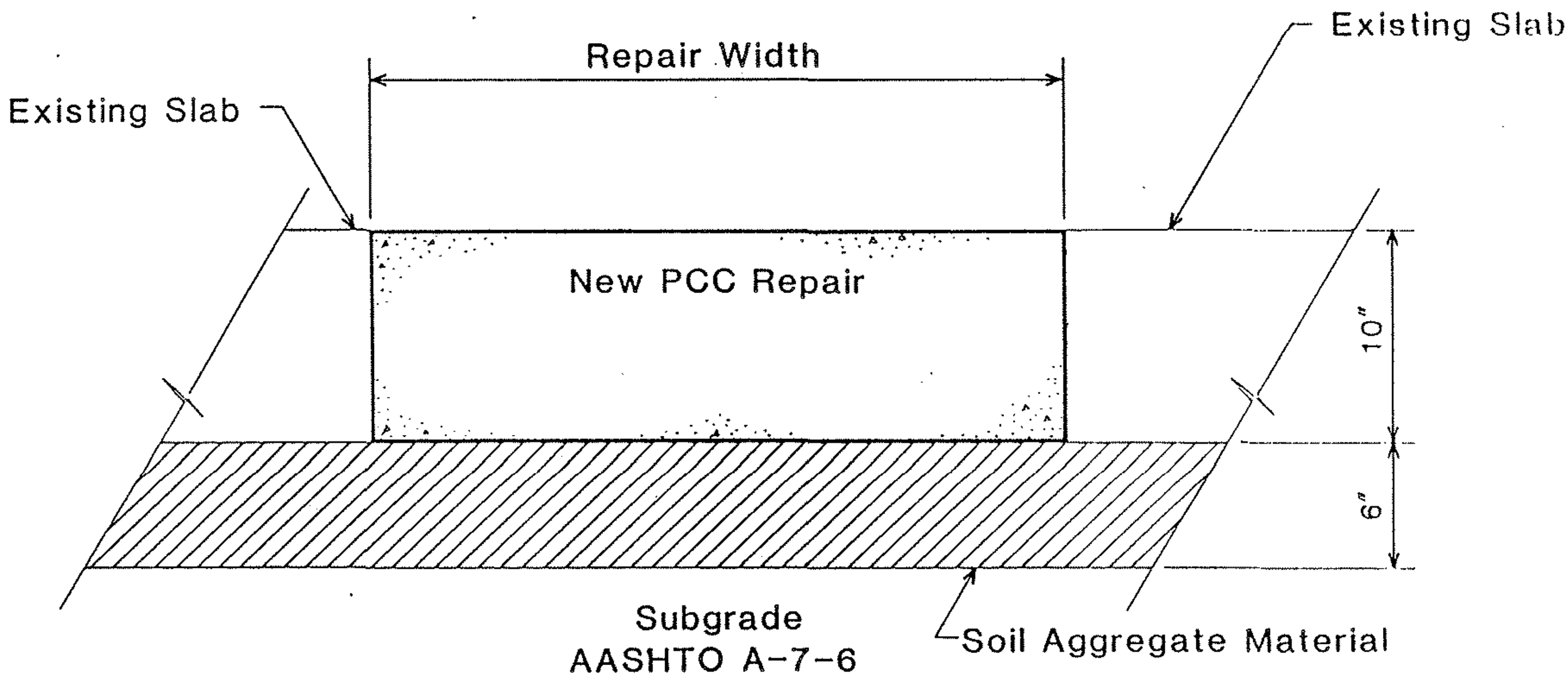


Figure 81. Illustration of Full-Depth Repair Design Used on NE080404 [1 in = 25.4 mm].

The original contraction joints have faulted significantly under heavy traffic and the repair joints are also displaying very high faults (despite having been in service only one year). This large repair joint faulting may be attributed to the lack of dowel bars in the full-depth repair design and the fine-grained subgrade present. Additionally, the repair leave joints consistently display higher faults than the repair approach joints.

The repair joint widths averaged 0.63 in [16.1 mm]. While the joint sealant is intact in all of the repair joints, it is often partially extruded, probably due to poor sealing techniques. Several of the repairs were cracked (see Figure 82). In addition, two of the three repair areas exhibited some transverse and longitudinal slab cracking.

Considering the short period of service (one year at the time of survey), the full-depth repairs placed in 1984 are not performing well. The major problem is the large joint faulting present on this pavement. Measurements of 0.20 in [5 mm] were common and one repair leave joint fault was 0.33 in [8 mm]. The rapid development of faulting on these repairs indicates that pumping is taking place. Improved joint sealing techniques, the use of mechanical load transfer devices, and possibly the concurrent use of slab stabilization techniques could have reduced these faults. If this rehabilitation were to be part of a complete pavement restoration program that includes diamond grinding of at least the driving lane, the above rehabilitation techniques would probably be cost-effective.

Full-depth repairs did successfully address cracking of the original pavement. However, a few transverse and longitudinal cracks have developed in the repairs, which indicate premature repair failure after only one year of service. These cracks may be due to the relatively short curing time (as little as four hours) allowed before traffic flow was restored.

Overall, this rehabilitation project is considered unsuccessful because of the unacceptable full-depth repair performance (excessive faulting). However, the pavement has several years of serviceable life remaining because slab cracking is relatively infrequent. Additional full-depth repairs should be placed and then the pavement should be diamond ground to provide a smooth riding surface.

While both NE080404 and NE080382 carried approximately the same traffic levels, NE080404 performed better (although rougher) because the exclusion of reactive aggregate reduced spalling and the shorter joint spacing produced fewer cracked slabs.

OH050022 - The original pavement was a 9-in [229 mm] JRCP with contraction joints sawed at intervals of 60 ft [18.3 m] and 100 ft [30.5 m]. The pavement was constructed and opened to traffic in 1954. Typically 4-ft [1.2 m] wide patches were placed in 1970 at approximately 70% of the joints to address joint spalling and faulting. Additional full-depth repairs were placed in 1984, but very few of these repairs were surveyed. All repairs were skewed 1:6 from the centerline to the shoulder (see Figure 83 for the design and layout of the repairs).

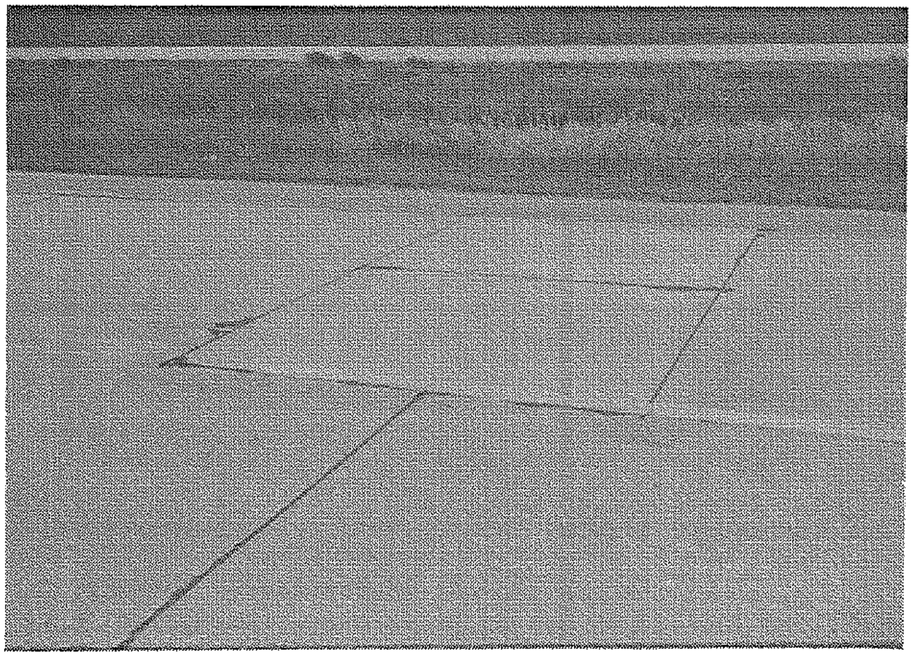
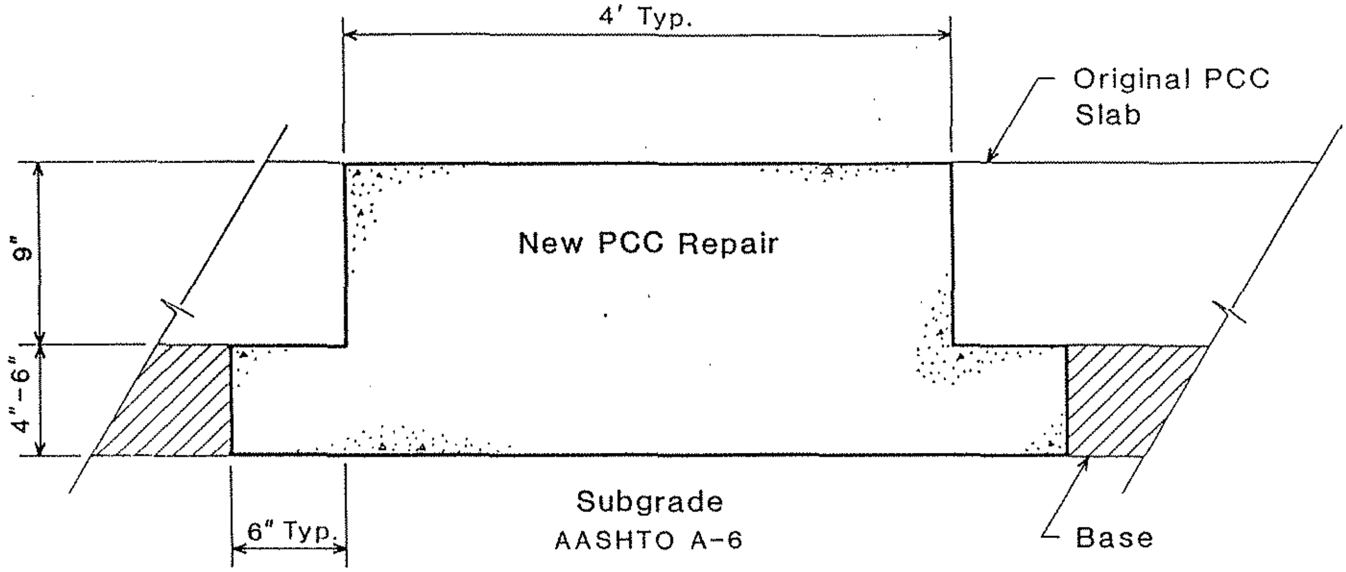


Figure 82. Photo of Full-Depth Repair on NE080404.



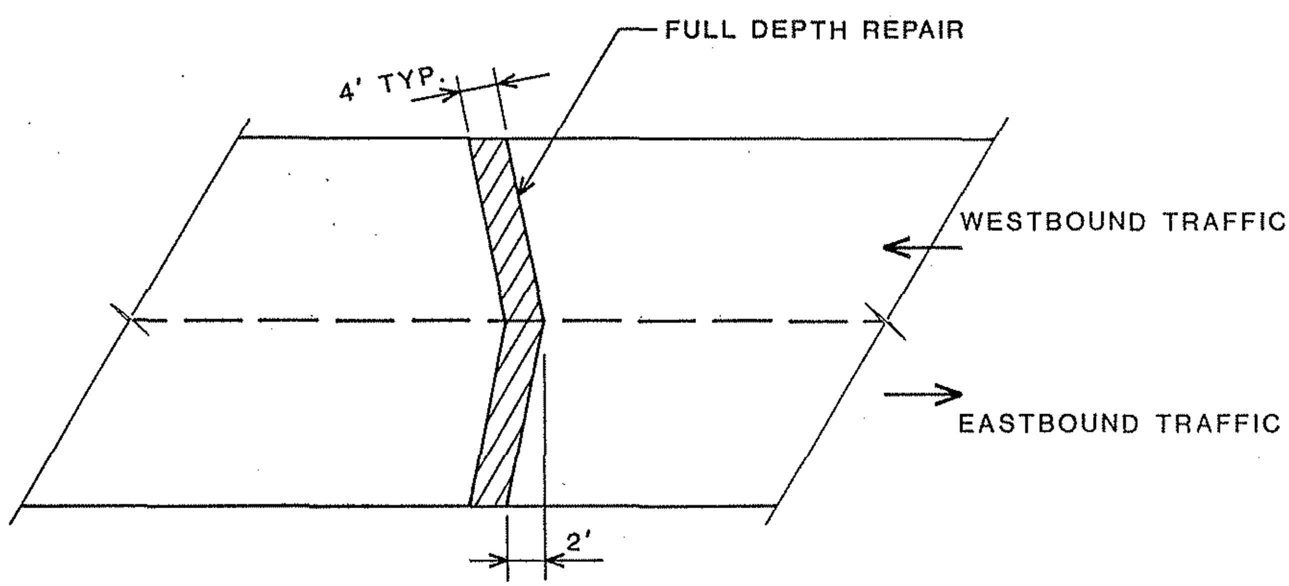


Figure 83. Illustration of Design and Layout of Full-Depth Repairs Used on OH050022 [1 in = 25.4 mm].

The following table summarizes faulting of the surveyed joints [1 in = 25.4 mm]:

	REG CONT 60´	REG CONT 100´	REP APP 60´	REP APP 100´	REP LV 60´	REP LV 100´
Total Meeting Acceptance Criteria	83%	77%	85%	90%	68%	75%
Average Fault	0.15	0.15	0.10	0.10	0.19	0.16
ESALs Sustained By Joint Type (millions)	4.0	4.0	2.4	2.4	2.4	2.4

Repair leave joints are larger than repair approach joints, and all joint types (including regular contraction joints) exhibited high faults. Although there were no visible signs of pumping, it is believed that pumping is occurring, probably due to poor consolidation of the undercut lip or poor consolidation of the subbase under the repair. Most of the repairs are raised above the original pavement, which would indicate that overfilling of the repair is also a cause of faulting.

Some corner spalls were observed near the shoulder area and where repair boundaries in adjacent lanes were not collinear. A few repairs exhibited major pressure spalls, but most of the spalling occurred outside of the repair. The westbound lane was spalled slightly more than the eastbound lane, but there appeared to be no difference in spalling between the different joint spacings. Longitudinal joint spalling was observed periodically throughout the surveyed sample units at repair locations. A few of the repairs surveyed exhibited some low-severity transverse and longitudinal slab cracking.

Good joint sealant practices may improve the future performance of the repairs. Sealant will keep water from penetrating to the subbase material and softening it. The sealant will also prevent incompressibles from intruding into the joint, thus preventing spalling.

Load transfer provided by dowels would be expected to reduce faulting of the repairs, if the faulting is not caused by overfilling. The undercut design, if constructed directly on the subgrade, would be highly susceptible to pumping and/or frost heave. In addition, there is the problem of achieving proper compaction of the subbase and consolidation of the repair material in the undercut lip.

Overall, this rehabilitation project is considered successful due to the long-term performance of the full-depth repairs (the performance data represents fifteen years of traffic, roughly 2.1 million ESALs). Considering the current pavement condition and traffic levels, the expected remaining life of this pavement is approximately three to five years from the date of survey (0.6 to 1 million ESALs). Thus major rehabilitation, in the form of additional full-depth repairs and diamond grinding or full-depth repairs and an overlay, is expected to be needed as early as 1988 or as late as 1990.

In considering the degree of success of this rehabilitation project, the low traffic levels must be considered. In fifteen years of service, the rehabilitated pavement received 2.1 million ESALs, which is what many Interstate pavements receive in just a few years. Although acceptable on this project, the performance provided by these repairs would probably be unacceptable on high-volume highways.

OH077053 - The original pavement was a 9-in [229 mm] JRCP with 60.0-ft [18.3 m] contraction joints and was constructed and opened to traffic in 1967. The project was an experimental section which provided a field study of the techniques developed in NCHRP 1-21, specifically full-depth repairs, partial-depth repairs, undersealing, load transfer restoration, diamond grinding, and joint resealing (37). All of these rehabilitation techniques were performed in the outer lane only.

Two full-depth repairs (one 22 ft [6.7 m] long and one 5 ft [1.5 m] long) were placed at two joints in 1982 to address joint spalling, joint faulting, and slab cracking of the original pavement. The repairs contained 1.25-in [32 mm], 18-in [457 mm] long dowels (3 in each wheelpath), and welded wire fabric reinforcement (0.23-in [5.8 mm] transverse wires on 12-in [305 mm] centers and 0.25-in [6.3 mm] longitudinal wires on 6-in [152 mm] centers). The joints were sawed and sealed after placement. The repairs were opened to traffic 5.5 hours after placement. The construction practices for the full-depth repairs were very poor. The design of the full-depth repairs is shown in Figure 84.

The following table summarizes faulting of the surveyed joints [1 in = 25.4 mm]:

	REG CONT (Non-subsealed)		REG CONT (Subsealed)		REP APP		REP LV	
	outer	inner	outer	inner	outer	inner	outer	inner
Total Meeting Acceptance Criteria	63%	100%	92%	N/A	50%	N/A	0%	N/A
Average Fault	0.19	0.10	0.13	N/A	0.23	N/A	0.24	N/A
ESALs Sustained By Joint Type (millions)	7.0	1.0	7.0	1.0	1.2	N/A	1.2	N/A

Faulting of the repair joints is very large and generally not within acceptable limits. The repairs appear to have been overfilled.

The two full-depth repairs placed in 1982 are performing marginally well. The smaller full-depth repair is in good condition, but the larger full-depth repair is exhibiting low-severity transverse and longitudinal cracking. In addition, the joint sealant is missing on this repair, and incompressibles have infiltrated the joints. The presence of incompressibles might have restricted movement of the repair, causing the slab to crack.

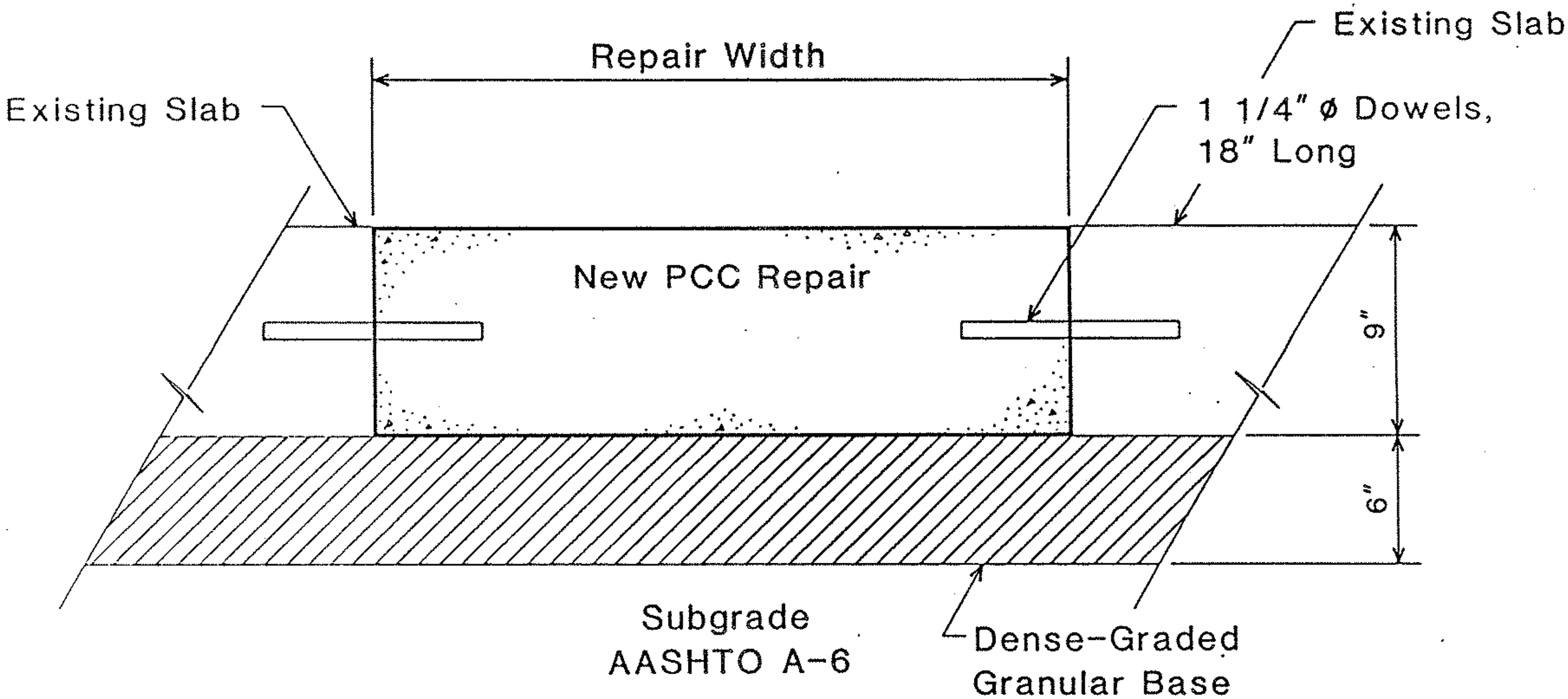


Figure 84. Illustration of Full-Depth Repair Design Used on OH077053
[1 in = 25.4 mm].

Contraction joint widths averaged 0.25 in [6.3 mm] for the repair approach joint and 0.15 in [3.8 mm] for the repair leave joint. Where spalling was observed it was of low severity. Longitudinal joint spalling was not observed in the repairs.

This repair technique successfully addressed the spalling problems, but it should be noted that the larger repair is unsealed, and it could spall again and deteriorate quickly. It now exhibits compression cracks at each end (probably from the incompressibles that are present), as well as a large transverse crack. It is possible that a support problem may exist at this location which has yet to be corrected.

Considering the present pavement condition and the current traffic loadings, the expected life of this pavement is estimated at approximately five to seven years from the installation of the full-depth repairs (4.0 to 5.6 million ESALs). Thus major rehabilitation, in the form of additional full-depth repairs and diamond grinding, is expected to be needed as early as 1987 or as late as 1989. The major problems to be addressed are the regular contraction joint faulting and the distressed full-depth repair.

VA044000 - The original pavement was a 9-in [229 mm] JRCP with 61.5-ft [18.7 m] contraction joints and was constructed and opened to traffic in 1967. Typically 4-ft [1.2 m] wide full-depth repairs were placed in 1984 at approximately 31% of the joints to address joint spalling (caused by Unitube joint forming inserts) and faulting. The repairs did not contain dowels. The concrete was formed against a preformed bituminous expansion joint filler (0.5 in [12.7 mm] thick) on the approach side. Figure 85 gives the design of the full-depth repairs on this project. Other rehabilitation techniques applied included partial-depth repairs, pressure relief joints, and joint resealing. Due to high traffic levels, only the outer lane was surveyed.

The following table summarizes faulting of the surveyed joints [1 in = 25.4 mm]:

	REG CONT		REP APP		REP LV	
	outer	inner	outer	inner	outer	inner
Total Meeting Acceptance Criteria	100%	100%	94%	N/A	47%	N/A
Average Fault	0.03	0.03	-0.10	N/A	0.24	N/A
ESALs Sustained By Joint Type (millions)	4.4	1.4	0.4	N/A	0.4	N/A

Faulting measurements for the repair approach joints were generally well within acceptable limits, but faulting measurements for the repair leave joint were often unacceptable. Many of the repairs were tilted (approach end low). After nearly 18 years of service, the regular contraction joints exhibit relatively little faulting, probably due to the good subbase present.

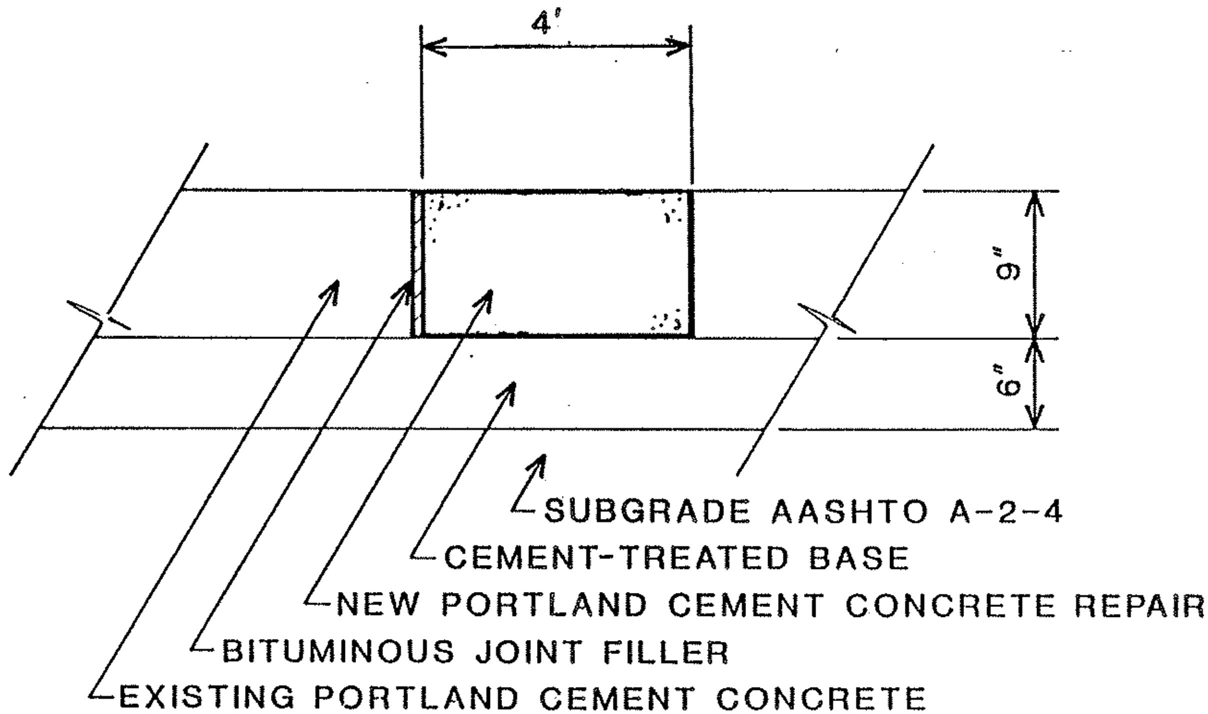


Figure 85. Illustration of Full-Depth Repair Design Used on VA044000 [1 in = 25.4 mm].

The repair joints were well sealed and free of incompressibles, but occasionally had extruded sealant (see Figure 86). There were no spalling or cracking problems associated with the repairs. Repair joint widths ranged from 0.5 in to 0.9 in [13 mm to 23 mm].

The full-depth repairs successfully addressed the joint spalling of the original pavement, and thus have prevented further deterioration of the pavement and joints. However, faulting of many of the repairs is a problem after only one year of service. The faulting data indicates that the absolute magnitude of faulting is almost always 0.1-0.2 in [3-5 mm] greater at the leave repair joint. This indicates that some actual pumping-related faulting is taking place--probably around 0.05-0.1 in [2-4 mm] so far.

Improved full-depth repair performance might have been achieved through the following steps:

1. Better hot-poured sealant installation practices (to prevent extrusion).

2. Inclusion of mechanical load transfer devices in the full-depth repairs to reduce faulting.

The section of this project with the full-depth repairs is structurally sound, but will require diamond grinding in the near future to remove existing faults at the repairs.

VA064202 - The original pavement was a 9-in [229 mm] JRCP with 61.5-ft [18.7 m] contraction joints and was constructed and opened to traffic in 1966. Two separate full-depth repair programs were performed in 1976 and 1984 at approximately 23% of the joints to address transverse joint spalling (caused largely by the Unitube joint forming inserts) and faulting. Only one 1976 full-depth repair was surveyed. Partial-depth repairs, pressure relief joints, and joint resealing were also performed.

The following table summarizes construction differences between the two repair years [1 in = 25.4 mm]:

Repair Year	Cement Type	Preformed Expansion Filler in Repair Approach Joint	Load Transfer	Drains
1976	Calcium Aluminate	0.5 in thick	Two Undercut Designs Used	No
1984	Type III	0.5 in thick	None	T-verse Agg

Figure 87 illustrates the design of each full-depth repair year. Smaller, partial-lane-width full-depth repairs were also placed in 1976 and 1984 to address smaller joint spalls (minimum size of 2 ft [0.61 m] by 2 ft [0.61 m]).

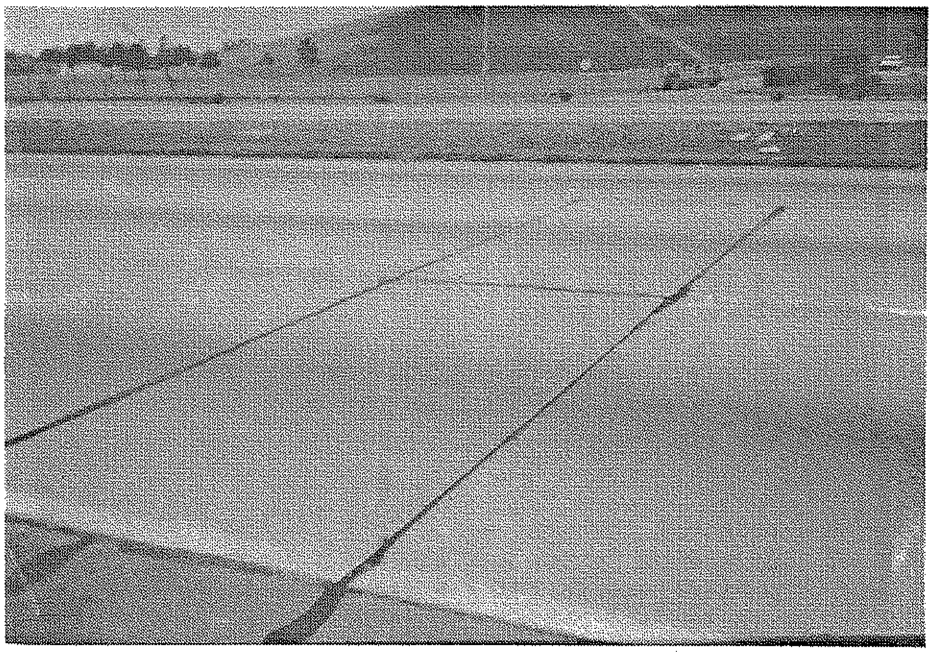


Figure 86. Photo of Typical Full-Depth Repair on VA044000.

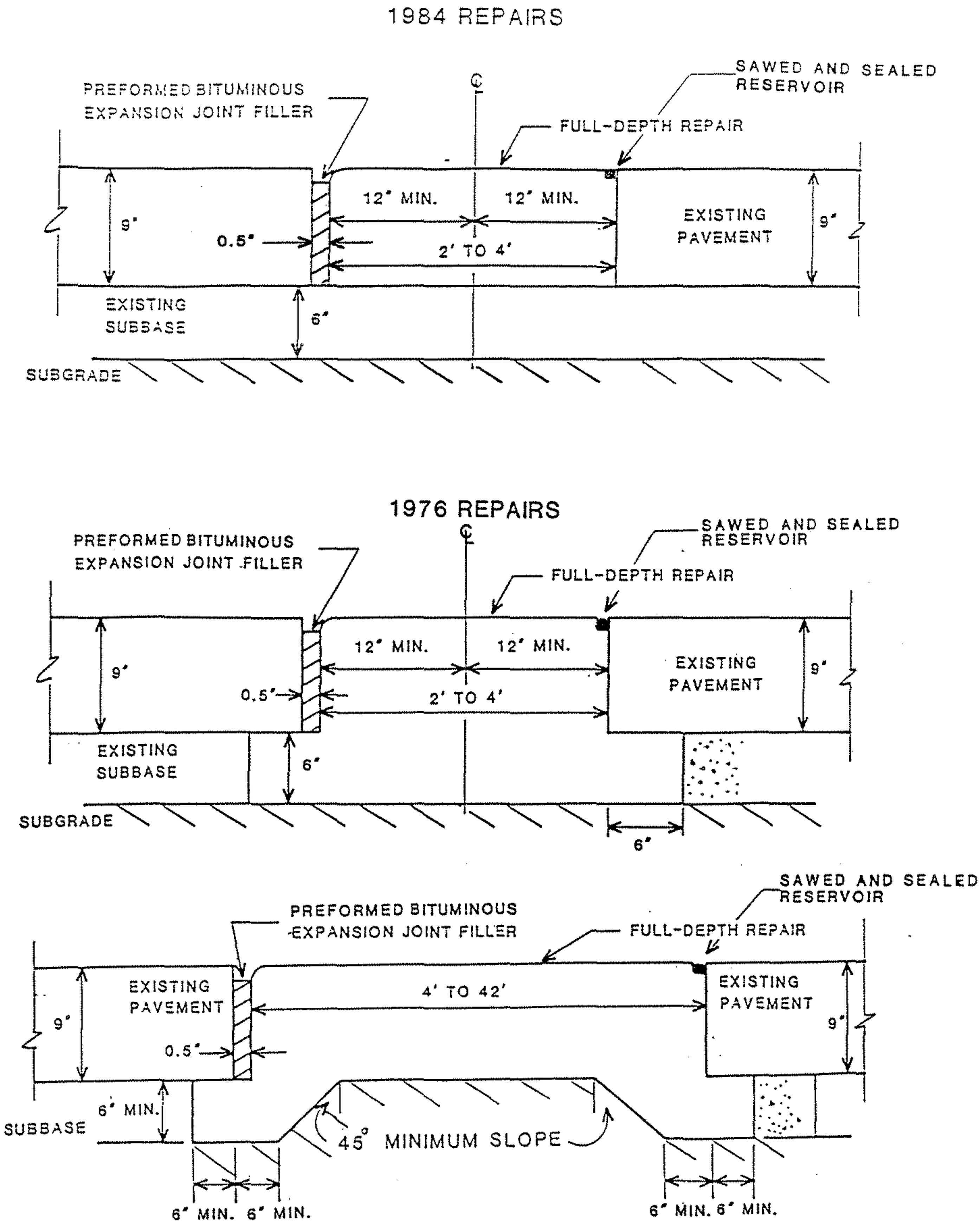


Figure 87. Illustration of Full-Depth Repair Designs Used on VA064202 [1 in = 25.4 mm].

The following table summarizes faulting of the surveyed joints [1 in = 25.4 mm]:

	1984 REPAIRS				1976 REPAIRS			
	REP APP		REP LV		REP APP		REP LV	
	outer	inner	outer	inner	outer	inner	outer	inner
Total Meeting Acceptance Criteria	100%	100%	33%	0%	100%	N/A	0%	N/A
Average Fault	0.09	-0.04	0.29	0.27	0.04	N/A	0.62	N/A
ESALs Sustained By Joint Type (millions)	0.4	0.1	0.4	0.1	3.4	N/A	3.4	N/A

Repairs in the first sample unit were generally faulted much less (leave joint average = 0.12 in [3 mm]) than those in the second sample unit (leave joint average = 0.45 in [11 mm]), which were located approximately at grade. Most of the repairs were either tilted (approach end low), indicating typical pumping action, or raised, possibly indicating soil heaving or overfilling of the repair. However, even the raised repairs were observed to have much higher faults on the leave side, suggesting that pumping was taking place on these as well.

The full-depth full-width repairs placed in 1984 are not cracked or spalled, and the joints remain generally well sealed and free of incompressibles after one year of service. The 1976 repair was cracked, missing its joint sealant, and the joints contained incompressibles. Medium-severity spalling was found on the approach side of this repair. Repair joint widths range from 0.1 in to 1.1 in [2.5 mm to 28 mm]. Good performance was found in the smaller, partial-lane-width full-depth repairs (minimum size 2 ft x 2 ft [0.61 m x 0.61 m]) for both repair years. The repairs are uncracked and in good condition.

The full-depth repairs placed in 1976 and in 1984 addressed spalling due to the use of Unitube metal joint forming inserts. It appears that the repairs were placed in time to repair this distress and prevent further spalling. Faulting remains a problem, particularly at the repair leave joints, even after only one year of service. It was noted that the repairs that were placed in 1984 (without load transfer devices) have faulted nearly as much as the one placed nine years ago using the undercut design.

It seems likely that the design of the full-depth repairs, which provide no load transfer in an area of long joint spacings, high moisture and relatively heavy traffic, is inappropriate. Since previous faulting data for the undercut repairs is unavailable, it is not known whether they developed faulting at an equally rapid rate. Elimination of the expansion joints and the incorporation of some type of positive load transfer system (e.g., dowels) could have improved repair performance.

Although only one 1976 repair was surveyed and comparative performance data was not obtained, it is suspected that the undercut design which contained the granular material would contribute to less pumping and faulting due to better drainage beneath the repair.

Full-depth repairs may have been detrimental to the overall performance of this project. The spalls produced by entrapment of incompressibles in Unitube joint inserts are generally confined to the top few inches of the slab and can often be addressed with partial-depth repairs. Although partial-depth repairs are more expensive on an area basis, smaller areas are addressed and the repairs are often more reliable because the original pavement load transfer system is left undisturbed. Faulting is unlikely to increase rapidly and existing and built-in roughness can be easily removed with diamond grinding.

Overall, this rehabilitation project is considered only partially successful. It was demonstrated that partial-depth repairs could provide good long-term (up to nine years or more) solutions to shallow joint spalling problems. Partial-width full-depth repairs were also used successfully to address these same problems. However, the full-width full-depth repairs placed in 1984 have faulted excessively.

Rehabilitation is needed <u>now</u> to return this pavement to a more serviceable state and prevent further deterioration. This rehabilitation should include load transfer restoration at the full-depth repair joints, diamond grinding to reduce pavement roughness, and joint resealing.

VA064279 - The original pavement was a 9-in [229 mm] JRCP with 61.5-ft [18.7 m] contraction joints, constructed and opened to traffic in 1967. One direction of the project was diamond ground while the other direction was not. Typically 4-ft [1.2 m] wide repairs were placed in 1981 at approximately 81% of the joints to address joint spalling caused by Unitube joint-forming inserts. The repair did not contain dowels, but selected repairs included transverse aggregate drains. The concrete was formed against a preformed bituminous expansion joint filler (0.5 in [12.7 mm] thick) on the approach side. The typical full-depth repair design for this project is the same as shown in Figure 85. Other rehabilitation techniques applied to the project include pressure-relief joints and joint resealing. The project has three lanes in each direction, but due to high traffic levels, only the outer two lanes were surveyed.

The following table summarizes the faulting on this project [1 in = 25.4 mm]:

	UNGROUND				DIAMOND GROUND			
	REP APP		REP LV		REP APP		REP LV	
	outer	center	outer	center	outer	center	outer	center
Total Meeting Acceptance Criteria	77%	94%	24%	0%	100%	100%	94%	70%
Average Fault	0.14	0.09	0.44	0.40	0.05	0.06	0.12	0.18
ESALs Sustained By Joint Type (millions)	2.8	1.6	2.8	1.6	1.4*	0.8*	1.4*	0.8*

* Since Grinding

As would be expected, the diamond ground lanes were smoother than the unground lanes. Faulting on the repair leave joints was generally very large, with a faulting configuration which would be expected if pumping exists. Significant faulting of the repair joints has taken place within the last year. It is unclear why the faulting of the center lane leave joint is larger than that of the outer lane. The repair joints are well sealed and free of incompressibles, but there are some locations where low-severity spalling is present.

The full-depth repairs placed in 1981 addressed the joint spalling of the original pavement. While the spalling was successfully repaired and the repairs have performed well structurally, it is evident that faulting problems persist in spite of the inclusion of transverse drains with some of the repairs. The diamond grinding performed in 1984 reduced the faulting, but it has continued to develop. All of the required ingredients for pumping/faulting are present at this site: high moisture levels, heavy traffic, erodible base/subbase materials, and joints with poor load transfer capability. Since good joint sealant maintenance seems to have been performed on the pavement and drainage design has been addressed (at least in the vicinity of the repairs), it seems that the most likely area of design improvement would be the provision of load transfer across the transverse joints.

Another possibility would have been to place partial-depth repairs to address the spalls caused by the Unitube metal inserts. This would have avoided disturbing the existing load transfer devices and might have reduced the levels of faulting now being observed.

In most cases, repair leave joints exhibited much greater faults than repair approach joints. One reason for this might be that many of the repairs were overfilled at placement, resulting in an initial positive fault for the leave joint and negative fault for the approach side. Another possible explanation might be that when the wheels cross the repair approach joint, free water is moved both backwards (resulting in approach joint faulting) and further forward as the repair tilts, since it is not long enough to act as a long slab on grade. The water that is moved forward is ejected back underneath the repair (and not forward at all) when the wheel crosses the repair leave joint, resulting in leave joint faults. Since the volume of water ejected under the leave joint is likely higher than that under the approach joint, the faulting that develops is correspondingly higher as well.

Overall, the full-depth repairs installed on this rehabilitation project are considered unsuccessful since they have increased pavement roughness considerably. The full-depth repair joints have, in fact, faulted more than twice as much in four years as the original contraction joints have in 18 years.

Suggestions for improvements include the incorporation of load transfer devices in future full-depth repairs, installation of subdrains along the entire project, and regrinding the pavement when roughness becomes unacceptable.

Considering the current pavement condition, traffic levels and rates of deterioration, the expected life of this pavement is approximately 2 to 4 years from the date of survey (1.8 to 3.6 million ESALs in the outer lane). The unground section would benefit from more immediate attention. With proper rehabilitation, this project should provide many additional years of service because it shows little evidence of fatigue or structural failure.

VA064284 - The original pavement was a 9-in [229 mm] JRCP with 61.5-ft [18.7 m] contraction joints and was constructed and opened to traffic in 1968. Two separate full-depth repair programs were performed in 1978 and 1984 encompassing all but one of the surveyed original contraction joints. The full-depth repairs addressed transverse joint spalling caused by Unitube joint forming inserts. Due to high traffic levels, only the outer lane was surveyed. The following table summarizes construction differences between the two repair years [1 in = 25.4 mm]:

Repair Year	Cement Type	Preformed Expansion Filler in Repair Approach Joint	Load Transfer	Drains
1978	Calcium Aluminate	0.5 in thick	1 in wide, 1/4 in thick Anchor Bars	T-verse Agg
1984	Type III	0.5 in thick	None	T-verse Agg

The designs of the full-depth repairs used in each year are illustrated in Figure 88.

The following table summarizes the faulting on this project [1 in = 25.4 mm]:

	1978 REPAIRS		1984 REPAIRS	
	REP APP outer	REP LV outer	REP APP outer	REP LV outer
Total Meeting Acceptance Criteria	96%	36%	91%	17%
Average Fault	0.08	0.32	0.10	0.30
ESALs Sustained By Joint Type (millions)	4.3	4.3	0.8	0.8

Faulting is a problem on both sets of repairs, especially on the repair leave joints. All of the 1978 repairs were tilted (approach end low) with respect to the original pavement, which is indicative of pumping. The 1984 repairs were faulted to about the same extent as the 1978 repairs. Pumping was identified as a stain on the shoulder near several of the 1984 full-depth repairs.

The 1978 repairs exhibited extensive longitudinal slab cracking, spalling and scaling. The joints are in very poor condition, with the sealant either split or absent, and incompressibles have infiltrated the

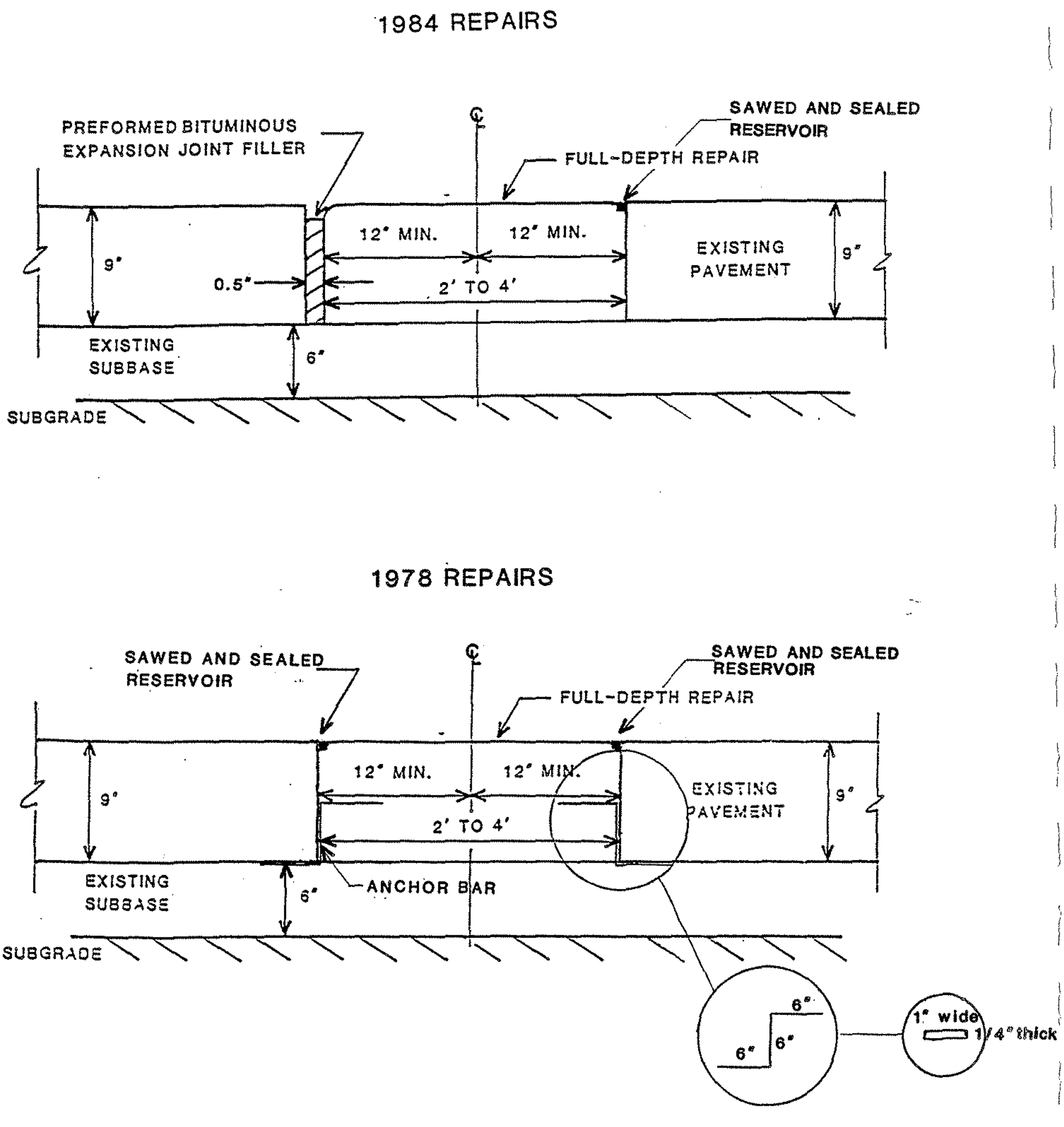


Figure 88. Illustration of Full-Depth Repair Designs Used on VA064284 [1 in = 25.4 mm].

joints. Nearly one half of all of the joints, with a slightly higher rate of occurrence at the approach joint, exhibited spalling. The adjacent slab was spalled only about one third as frequently, as seen in Figure 89. Contraction joint widths ranged from 0.6 in to 0.9 in [15.2 mm to 22.9 mm]. The 1978 repairs are slated for replacement, as evidenced by sawcuts outside the repairs.

The transverse contraction joints associated with the 1984 full-depth repairs are still well sealed, free of incompressibles, and unspalled. They are, however, quite faulted. Contraction joint widths ranged from 0.0 in to 0.8 in [0.0 mm to 20.4 mm].

The full-depth repairs placed in both years addressed the joint spalling and faulting of the original pavement. These same problems are now present in many of the repair joints, although the source of the spalling is no longer the metal joint inserts that were used in the original pavement.

The 1978 repair cracking may be due to one or more of the following:

1. The very stiff cement-treated subbase, which induces higher stresses in a curled or warped repair than a granular subbase would.

2. Expansion of the original pavement slab during periods of hot weather, producing longitudinal cracking in short slabs constructed of weaker material.

3. The calcium aluminate cement used in the repairs, which is known to undergo chemical conversion at temperatures above 77°F [25°C], causing considerable strength loss in the concrete.

4. Inadequate curing conditions (time, moisture retention, etc.).

Many of the repairs exhibit two longitudinal cracks which divide the repairs approximately into thirds. This would be expected if the repair curled (edges down) on a hot, sunny day and was loaded by traffic and its own weight. The anchor bars that were used would not sufficient to restrain the curling and the cement-treated subbase would be stiff enough to allow the repair to rest on its edges. The center of the repair would then be largely unsupported and susceptible to high stresses and cracking.

The higher incidence of spalling within the 1978 repairs than on the adjacent slabs indicates that the repair material may be weaker than the original slab. The closure of the pressure relief joints suggests that slab movement has taken place and may be continuing. If this is the case, the weaker material in the repairs may be crushed during periods as the slabs expand in warm temperatures. Much of this spalling might have been eliminated with the use of wider joint sealant reservoirs and a more timely joint resealing program.

While the faulting of the 1978 repairs is largely unacceptable, it is not significantly more than that of the 1984 repairs, which had been in service for only one year at the time of survey. Since it is unlikely that both sets of repairs have reached the limit of their faulting potential,

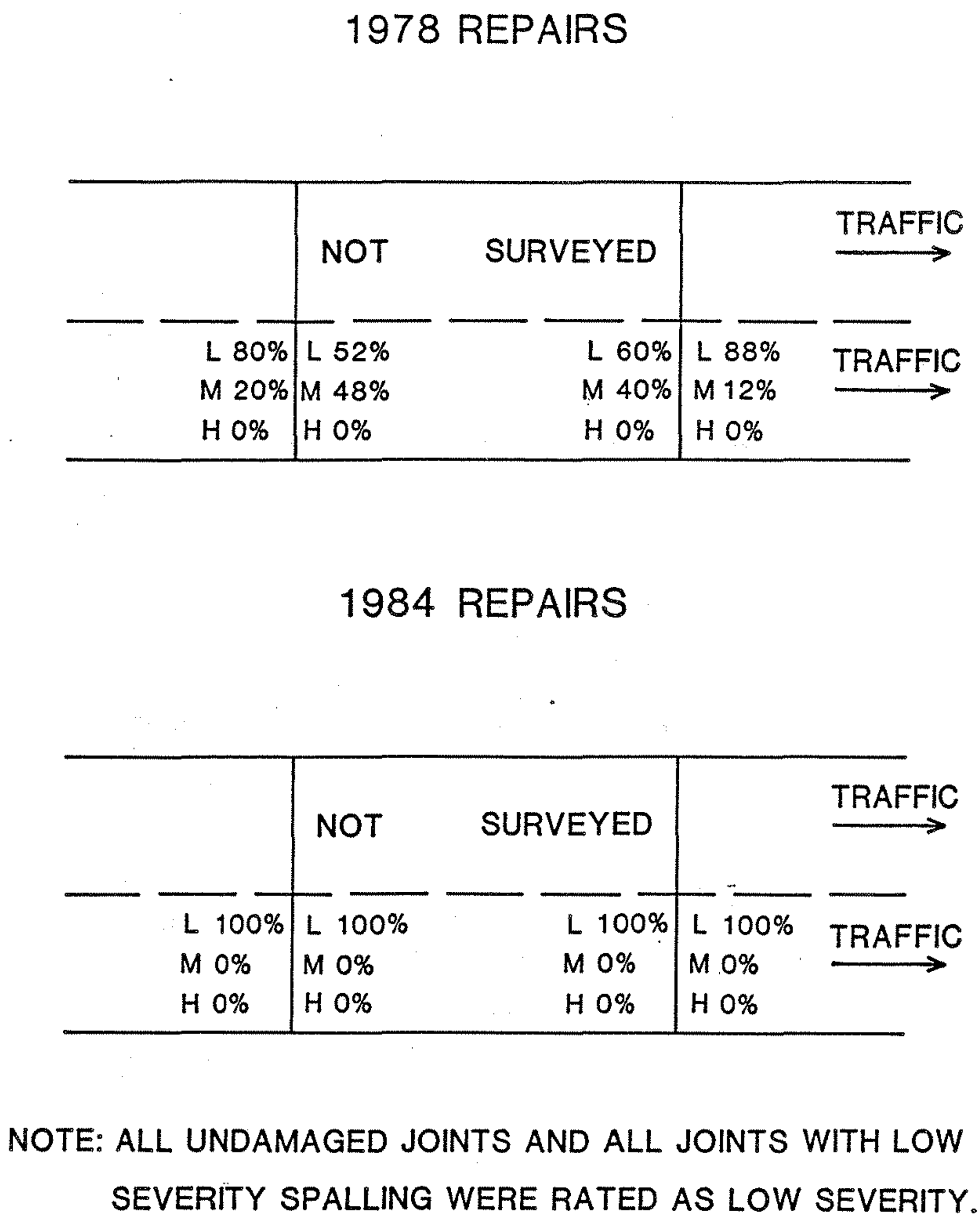


Figure 89. Summary of Spalling at Repair Locations on VA064289.

some aspect of the 1978 repair design must be accountable. It is likely that the incorporation of drains and tight joints with a mechanical load transfer device (the anchor bars) made the difference, although the utilization of 0.25-in [6.4 mm] thick, 1-in [25.4 mm] wide steel bars on 24-in [0.6 m] centers as load transfer devices seems questionable, particularly because their effectiveness is one-directional.

Also, the suitability of the calcium aluminate cement as a repair material is questionable. Although calcium aluminate cement does have applications where high early strength is required, it is subject to chemical conversion and substantial strength loss at high temperatures.

The 1984 full-depth repairs have not developed any spalling or cracking problems, but they exhibit high joint faults that equal those of the 1978 repairs which have been in service for 7 years. As suggested above, the difference may be the lack of good load transfer across the repair joints because not only are mechanical devices not present but expansion joints have been provided at the 1984 repairs, thus precluding the possibility of aggregate interlock in all but the hottest weather. The use of aggregate drains might have also increased the faulting. If the curling theory is valid for the 1978 repairs, it might be expected that the 1984 repairs will also eventually exhibit longitudinal cracking.

Overall, this rehabilitation project is not considered completely successful, primarily due to the poor performance of the full-depth repairs. The 1978 repairs are currently in poor condition and are apparently about to be replaced. The 1984 repairs are displaying some unusually high faults for such new repairs.

This project demonstrates the need for load transfer devices on heavily trafficked pavements, even when excellent subbase and subgrade materials are present and particularly where expansion joints are to be included in the repair design. It also demonstrates the need for good pavement drainage, especially in the vicinity of full-depth repairs, and good joint sealing and maintenance practices.

The 1976 repairs are currently being rehabilitated, so these repairs lasted about 7 years. It is not expected that the 1984 repairs will last as long because of their already excessive faulting and the possibility of the development of longitudinal slab cracking due to high traffic and curling stresses. Within the next few years the project will probably become unacceptably rough and require rehabilitation through either installation of new repairs or grinding and load transfer restoration of the existing repairs.

VA095000 - The original pavement was a 9-in [229 mm] JPCP with 20-ft [6.1 m] contraction joints and was constructed and opened to traffic in 1963. Typically 4-ft [1.2 m] wide repairs were placed in 1984 at approximately 35% of the joints to address joint spalling (caused by Unitube joint-forming inserts) and faulting. The repairs contained no mechanical load transfer devices. Figure 85 gives the typical full-depth repair design used on this project. Other rehabilitation techniques performed include partial-depth repairs and joint resealing.

The following table summarizes faulting of the surveyed joints [1 in = 25.4 mm]:

	REG CONT		REP APP		REP LV	
	outer	inner	outer	inner	outer	inner
Total Meeting Acceptance Criteria	83%	98%	94%	100%	87%	100%
Average Fault	0.13	0.07	0.09	-0.16	0.13	0.08
ESALs Sustained By Joint Type (millions)	7.6	1.1	0.6	0.1	0.6	0.1

Faulting measurements for the repair joints compares favorably to those of the original transverse contraction joints. The repair leave joints are faulted more than the repair approach joints. Many of the repairs were tilted (approach end low), the configuration which often accompanies pumping. Significant pumping was noted at many of the repairs.

The repair joints are well sealed and free of incompressibles, and no slab cracking exists. There are no spalling or cracking problems associated with the repairs. Repair joint widths ranged from 0.2 in to 0.9 in [5 mm to 23 mm].

The full-depth repairs addressed joint spalling and faulting of the original pavement. While the joint spalling was successfully addressed by these repairs, the placement of the full-depth repairs (some of which incorporated expansion joints) seems to have allowed rapid development of localized pumping, opening of joints and loss of support. The presence of heavy truck traffic in a wet environment with erodible subgrade materials would suggest the need for load transfer devices even on short-jointed pavements. The use of drains to remove excess water from the pavement structure might also have improved the performance of these repairs.

If the spalls addressed by these repairs were limited to the top 2-3 inches [51-76 mm] of the pavement, better performance might have resulted from the use of partial-depth repairs. While the initial cost of placing partial-depth repairs is sometimes higher than that of full-depth repairs (depending on the number and size of the repairs), their use in this case may have resulted in improved long-term serviceability due to reduced faulting. The placement of full-width partial-depth repairs to address entire joints (rather than small repairs to address individual spalls) would probably have been cost-effective.

The full-depth repairs placed to correct faulting and spalling are faulting and pumping badly and are beginning to crack. They will probably reach unacceptable levels of roughness within a year or two and will need to be replaced or rehabilitated with load transfer restoration, subsealing and diamond grinding. Improved drainage throughout the project might have improved the performance of these repairs (together with load mechanical load transfer). It is also possible that the repairs were unnecessary and that the spalling could have been handled more effectively with partial-depth repairs.

VA081147 - The original pavement was 9-in [229 mm] JRCP with 61.5-ft [18.7 m] contraction joints and was constructed and opened to traffic in 1965. Typically 4-ft [1.2 m] wide patches were placed in 1984 to address faulting, as well as joint spalling caused by Unitube joint forming inserts. Two different undercut designs were used (as shown in Figure 90), but one dowelled repair was also surveyed. In addition, load transfer restoration (shear devices and retrofit dowels) were placed at two repairs (not undercut) as a demonstration of the products. Other rehabilitation techniques applied include partial-depth repairs, undersealing, diamond grinding, underdrains, load transfer restoration, joint resealing, and pressure relief joints.

The following table summarizes faulting of the surveyed joints [1 in = 25.4 mm]:

	REG CONT		REP APP		REP LV	
	outer	inner	outer	inner	outer	inner
Total Meeting Acceptance Criteria	100%	100%	100%	100%	100%	100%
Average Fault	0.02	0.01	0.03	0.02	0.11	0.08
ESALs Sustained By Joint Type (millions)	8.8	1.2	0.8	0.2	0.8	0.2

Faulting measurements for the repair joints compare favorably with that of the original pavement. However, the repair leave joints are faulted more than the repair approach joints. The orientation of the repairs is such that it appears that pumping is occurring at the repairs, in spite of the undersealing that was performed.

The repair joints are well sealed and free of incompressibles, and no slab cracking exists. There are no spalling or cracking problems associated with the repairs. Contraction joint widths ranged from 0.5 in to 1.0 in [13 mm to 25 mm].

There appear to be nearly equal numbers of raised and approach-end-low repairs, but even the raised repairs are faulted more at the leave joint, which implies that pumping is taking place in spite of the undersealing that was performed in the vicinity of the repairs. Faulting measurements for the repair joints were always within acceptable limits and did not vary appreciably with repair load transfer design. The repair leave joints display particularly high faults, considering that they have only been in service for 1 year (sustaining 0.8 and 0.2 million ESALs in the outer and inner lanes, respectively), probably due to the absence of dowels in most of the repairs.

The shear devices used in one repair have failed in shear and tension along the bond between the matrix and the existing slab. The dowels in slots appear to be intact.

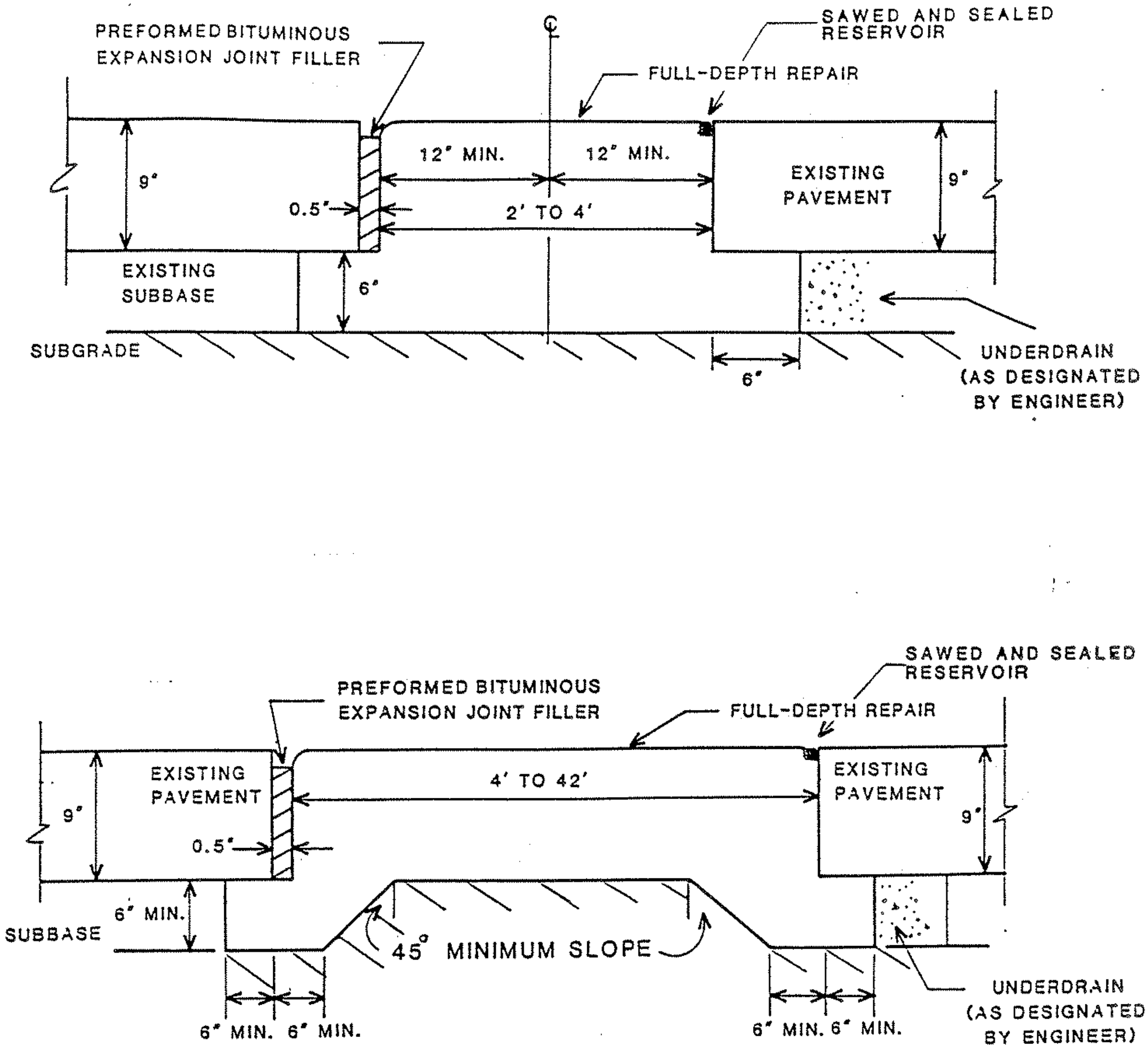


Figure 90. Illustration of Full-Depth Repair Designs Used on VA081147 [1 in = 25.4 mm].

The full-depth repairs placed in 1984 addressed severe slab cracking in the original pavement. Since the repairs have not cracked, it is assumed that the structural integrity of the pavement has been significantly improved and that the repair joints are much more easily maintained than working cracks. The successful performance of these repairs can probably be attributed in part to the use of cement grout undersealing and the placement of longitudinal drains in the vicinity of the repairs.

Faulting of the repairs is of some concern because only one year had passed since repair placement and diamond grinding were accomplished. While all of the repair faults are still within acceptable levels, they should be monitored for pumping and additional faulting.

The use of dowels in slots and shear devices in core holes has not significantly improved the short-term performance of these repairs. The shear devices appear to have failed in shear and tension at the bond between the matrix and the original slab.

This rehabilitation project is considered successful because the overall rate and level of deterioration has been reduced by the applied rehabilitation. While it is possible that localized areas of roughness will develop near the pressure relief joints and some of the full-depth repairs, it is expected that this pavement will provide good serviceability for 5 to 7 years from the date of survey (4 to 6 million ESALs), given the current pavement condition and traffic levels. Portions of the project were scheduled for CPR in 1985.

WV070002 - The original pavement was a 9-in [229 mm] JRCP with 61.5-ft [18.7-m] contraction joints and was constructed and opened to traffic in 1959. Full-depth repairs (undercut 6 in [152 mm] behind and beneath the existing slab) were installed in 1981, 1982, and 1983 at approximately 56% of the joints to address joint spalling, joint faulting, and slab cracking. Longitudinal underdrains were also placed in conjunction with the repairs. Figure 91 gives the design of the full-depth repairs on this project.

The following table summarizes faulting of the surveyed joints [1 in = 25.4 mm]:

	1983 REPAIRS				1982 REPAIRS			
	REP APP		REP LV		REP APP		REP LV	
	outer	inner	outer	inner	outer	inner	outer	inner
Total Meeting Acceptance Criteria	66%	84%	38%	40%	91%	85%	18%	39%
Average Fault	0.18	0.11	0.30	0.25	0.11	0.12	0.52	0.24
ESALs Sustained By Joint Type (millions)	8.0	2.5	8.0	2.5	13.0	5.0	13.0	5.0

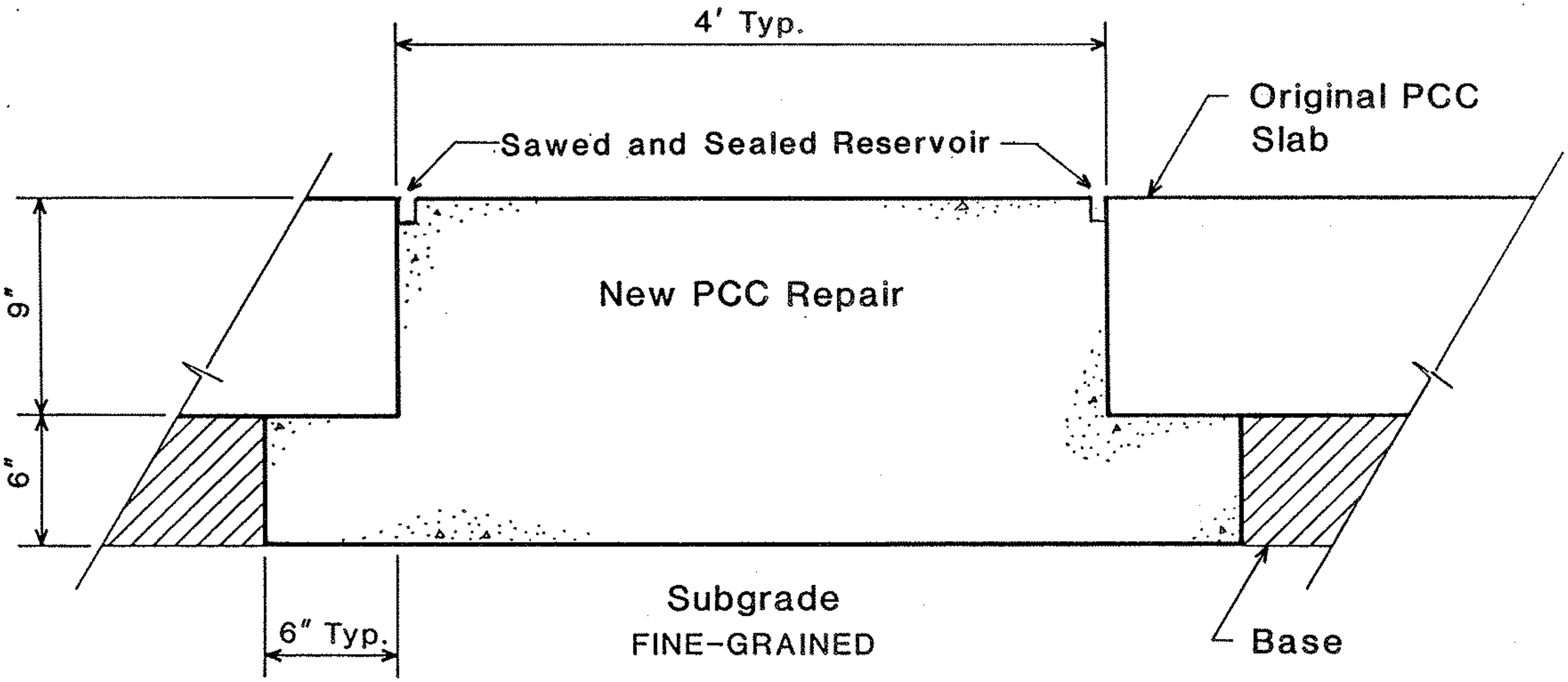


Figure 91. Illustration of Full-Depth Repair Design Used on WV07002
[1 in = 25.4 mm].

Faulting measurements on the leave side of both sets of repairs were very large and generally not within acceptable limits. The repairs were often tilted (approach end low), indicating typical pumping action. Pumping was observed with some of the repairs. However, some repairs were raised, which can be attributed to overfilling of the repair at construction or frost heave, since many of the repairs were probably constructed directly on the fine-grained subgrade. Most of the raised repairs were also tilted (approach end low), indicating the presence of pumping throughout the project.

In general, the full-depth repairs placed from 1981 to 1983 are performing poorly. They have faulted severely and provide a very rough riding surface. Several of the surveyed repairs exhibited low- and medium-severity transverse cracking (see Figure 92). These cracks tended to occur in longer (15 ft [4.5 m] or more) repairs and were spaced about 10 to 15 ft [3 to 5 m] apart. Thus, repairs 15 to 20 ft [4.5 to 6 m] in length typically exhibited one crack while longer repairs were cracked in more than one location. The longer repairs utilized an undercut design that did not remove the base underneath the center of the repair, thus making it easier to crack transversely. A few instances of medium-severity or greater longitudinal repair cracking were noted on shorter repairs. Given the thickness of these repairs, it is likely that these cracks are the result of compressive stresses rather than fatigue from heavy traffic and loss of support.

The effect of foundation conditions on repair performance is unclear. The 1982 repair leave joints (located cut areas) averaged only 0.23 in [5.8 mm] of faulting (only two such joints were measured), while similar joints located in areas of fill (ten joints measured) averaged 0.40 in [10.2 mm] of faulting. The 1983 repairs produced opposite results, with the leave joints located in fill (15 measured) averaging 0.20 in [5.1 mm] and those located in cut (13 measured) averaging 0.42 in [10.7 mm].

Most of the transverse contraction joints on both sets of full-depth repairs are performing poorly. While joint widths and spalling are acceptable, joint faulting has been severe, particularly across the repair leave joints. Repair joints have not opened appreciably, although incompressibles were sometimes present and a small amount of low- and medium-severity spalling was observed. Most of the higher-severity spalling occurred in the outer lanes. There was little difference in the amount and severity of spalling between the 1982 and 1983 repairs. Joint widths averaged approximately 0.3 in [7.6 mm].

The faulting of the repairs is due largely to the use of the inverted "T" load transfer design which allows differential vertical movement across the joints and provides the mechanism for the pumping. In addition, the undercut repairs required local excavation of much of the subbase and, as a result, the repairs were often placed directly on the fine-grained subgrade. Water infiltrating the transverse joints had a direct route to the subgrade, which also facilitated pumping and faulting. Properly installed dowels and elimination of the undercut would have resulted in much less pumping, as shown by the original contraction joints and the reconstructed contraction joints within the longer repairs, which exhibited relatively little faulting. The effectiveness of the longitudinal drains would also have been improved by leaving the granular subbase in place.

Figure 92. Photo of 1982 Full-Depth Repair with Slab Cracking and Spalling on WV070002.

Longitudinal and transverse cracking was a problem in many repairs. While the occurrence of longitudinal cracking was infrequent and restricted to shorter repairs, it is believed that it could be reduced or eliminated by improving joint sealing and maintenance activities to further restrict the entry of incompressible materials which result in points of high bearing stress. Restrictions on time of placement or ambient temperature might also reduce the risk of overstressing the repair in compression while it is gaining strength.

The use of longer repairs indicates that the original slab was badly cracked. The recurrence of these cracks indicates that either additional rehabilitation (base repair, improved drainage, etc.) should have been performed or the pavement was not cured sufficiently prior to reopening to such heavy traffic. Proper curing is essential on a heavily trafficked pavement such as this. A shorter repair slab design would have reduced the cracking and made the slab discontinuities easier to seal.

The effectiveness of these repairs can be evaluated from two viewpoints. The first is that the repairs have deteriorated to their present condition after enduring more traffic than many pavements experience in a lifetime. From this standpoint, their deterioration is understandable and they would be judged to have performed acceptably if they were in their present condition after 20 or more years of service on another pavement. However, the other viewpoint is that the repairs have not performed acceptably on the this pavement, where after only 2 or 3 years many of them require additional rehabilitation.

Overall, this rehabilitation project is considered unsuccessful because the full-depth repairs have cracked and faulted excessively over a short period of time. In view of the heavy traffic and wet-freeze climate at the project site, it is believed that properly installed dowelled repairs without undercuts would have provided improved repair performance and would have enhanced the effectiveness of the longitudinal underdrains. Repair cracking might have been reduced by reducing the repair slab lengths to 15 ft [6 m] or less and maintaining transverse joint seals.

Rehabilitation is currently needed throughout the project to correct roughness due to repair faulting and deteriorating cracks. This could be accomplished in the first two sample units by retrofitting load transfer devices across the repair joints, installing additional dowelled full-depth repairs where needed and diamond grinding the pavement. The third sample unit, which is more deteriorated than the other two, probably needs a structural overlay or complete reconstruction. The West Virginia Department of Highways has planned rehabilitation for this pavement for the fall of 1985.

CHAPTER VII

PANEL CONCLUSIONS AND RECOMMENDATIONS

PERTAINING TO THE DESIGN AND USE OF FULL-DEPTH REPAIRS

Conclusions

1. Full-depth repairs were observed to perform well from one year (1-2 million ESALs) to over ten years (2 to 6 million ESALs or more). This variation in performance can be attributed to design and construction factors.

2. Full-depth repair performance (faulting, spalling, and cracking) varies inversely with the volume and magnitude of heavy truck loads imposed during the life of the repair.

3. Limited data indicates that the use of more dowel bars and of larger-diameter dowel bars reduces faulting.

4. Repairs that contained mechanical load transfer devices (dowels) and were subjected to heavy truck traffic in wet areas displayed relatively low amounts of faulting and spalling <u>when the dowels were properly installed</u>.

5. Repairs that did not contain mechanical load transfer devices and were subjected to heavy truck traffic in wet areas displayed significant amounts of faulting.

6. Repairs that did not contain mechanical load transfer devices and were subjected to heavy truck traffic in dry areas displayed relatively low amounts of faulting. The presence of reactive aggregate and a permeable subgrade in many of these cases is also believed to have helped minimize faulting.

7. Undercut (undowelled) repairs exhibited significant pumping, faulting, and deflections and did not perform well under heavy truck traffic.

8. Shorter full-depth repairs (4 to 6 ft [1.2 to 1.8 m]) appear to be more susceptible to longitudinal cracking. Repair lengths of 6 to 10 ft [1.8 to 3.0 m] seem to perform well with few cracking problems. Longer repairs (greater than 15 ft [4.6 m]) were observed to have a tendency to develop transverse cracks at approximately mid-slab.

9. Calcium aluminate repairs did not perform well, exhibiting much slab cracking and spalling.

10. Sawing and sealing of full-depth repair boundaries substantially reduced the occurrence of spalling at these locations.

11. Poor dowel bar grouting techniques had an adverse effect on repair performance, resulting in loss of load transfer, rapid faulting, and deep spalling.

12. Some "faulting" of full-depth repairs is attributable to overfilling or underfilling the repairs so that the finished surface is slightly higher or lower than the adjacent slab.

13. The repair approach joints were often observed to be narrower than the repair leave joints. It is believed that the repair actually slides along the subbase in the direction opposite traffic as a result of the applied torque of passing vehicle wheels.

14. Repair leave joint faults were generally much larger than repair approach joint faults. Since the repair leave joints were also generally wider than the approach joints they could be expected to fault more because of reduced deflection load transfer due to lost aggregate interlock and higher dowel bearing stresses that accompany wider joint openings.

Recommendations

1. Full-depth repair design should consider the following major inputs:

   - truck traffic level
   - desired life
   - environmental conditions
   - existing transverse joint design and spacing
   - subgrade drainability
   - design of existing pavement
   - existence of "D" cracking or reactive aggregate in original slab
   - allowable lane closure time
   - location, extent, and severity of existing deterioration
   - performance history of various repair designs under similar conditions

2. Full-depth repair design should include specific recommendations for the following items (to be included in plans and specifications):

   - load transfer of transverse joints
     - -dowel diameter
     - -dowel spacing
     - -dowel anchoring
   - repair length, use of intermediate joints

- o removal of PCC and disturbance of foundation
- o repair thickness
- o curing, minimum strength at opening
- o sealant reservoir of transverse and longitudinal joints
- o type of transverse repair joint
- o subdrainage
- o suitable materials
- o tie along longitudinal joint for repairs greater than 15 ft [4.6 m]

3. Specific design recommendations for full-depth repairs include:

   - o load transfer of transverse joints

     -properly installed dowels provide the most effective means for load transfer to reduce faulting

     -dowels should be used for all repairs, except for expansive pavements or low truck traffic volumes

     -undercutting alone does not provide adequate load for high truck traffic volumes (the addition of dowels to undercut repairs has provided good performance)

     -dowel diameter should be at least 1.50 in [38 mm] for high truck traffic volumes (when dowels are the sole form of load transfer)

     -dowel spacing should be 12 in [305 mm] with a minimum of 5 dowels per wheelpath for high truck traffic volumes

     -dowel anchoring must be clearly specified and properly controlled

     -dowels should be corrosion-resistant

   - o minimum repair length should be 6 ft [1.8 m]; maximum repair length should be 15 ft [4.6 m] before requiring reinforcement or an intermediate joint

   - o repair thickness should be equal to or greater than slab thickness depending on truck traffic volumes

   - o where a drainage improvement is not required, the foundation should not be disturbed; if disturbed, fill with concrete

- a minimum compressive strength of 2000 psi [13.8 MPa] should be obtained before opening to traffic (minimum flexural strength of 300 psi [1.7 MPa], third-point loading)

- sealant reservoir of transverse and longitudinal joints shall be sawed (or formed) and sealed before opening to traffic to prevent spalling and water infiltration

- where an expansion joint is constructed as part of the repair, the joint width should be 1 in [25 mm] or less and spaced not less than 1000 ft [305 m] from the nearest pressure relieving feature (not recommended for pavements with reactive aggregate)

- subdrainage should be provided in wet climates with fine-grained soils and high truck traffic volumes

- repair should be finished level with the adjacent slab and to a similar texture

CHAPTER VIII

PARTIAL-DEPTH REPAIR BACKGROUND INFORMATION

Partial-Depth Spall Repair

Introduction

Partial-depth repair is an alternative to full-depth repair in areas where slab deterioration is located primarily in the upper third of the slab and the existing load transfer devices (if any) are still functional. Partial-depth repairs restore ride quality to pavements which have spalled joints near the surface. Partial-depth repair of spalled areas also restores a well defined, uniform joint sealant reservoir prior to resealing existing joints. When properly placed with durable materials, these repairs can perform for many years. Current design and construction specifications, procedures and guidelines for partial-depth repairs are presented in the Appendices to this report.

Appropriate Uses and Locations

Partial-depth repair can be used to address certain types of distress which affect only the top few inches of the slab. These distresses include (25, 30, 41, 58, 84, 85, 133):

1. Spalls caused by metal (Unitube) joint inserts.

2. Spalls caused by intrusion of incompressible materials into the top of the joint.

3. Localized areas of scaling.

4. Spall distress associated with the early stages of "D" cracking or alkali-aggregate reactivity.

Many of these distresses occur adjacent to joints. Effective sealing of these joints requires repair of the adjacent distress. Failure to repair these areas prior to placement of an overlay will often contribute to reflective cracks which break down rapidly, causing premature failure of the overlay.

If several spalls are present on one joint, it is usually more economical to partial-depth repair the entire length of the joint than to repair individual spalls. Small spall areas along joints generally do not require repair. Areas less than 6 in [152 mm] long and 1.5 in [38 mm] wide at the widest point are normally not repaired, but are filled with sealant (unless a preformed compression seal is to be used in the joint).

Material Considerations

Repair Materials
Repair material selection depends on available curing time before opening to traffic, ambient temperature, available funds, and the size and

depth of the repairs. High-quality Portland cement concrete is generally accepted as the most universally compatible repair material. Typical mixes combine Type I, II, or III Portland cement with a coarse aggregate not greater than one half the minimum repair thickness (3/8-in [10 mm] maximum size is often used). Type III Portland cement concrete or the use of set-accelerating admixtures are often specified if the concrete repair must be reopened to traffic quickly.

Type III cement, with or without admixtures, has been used for repair mixtures longer and more widely than most other materials because of its availability, relatively low cost, and ease of use. Rich mixtures (up to 8 bags) gain strength rapidly during warm weather, although the rate of strength gain may be too slow to permit quick opening to traffic in cool weather. Insulating layers can be used to retain the heat of hydration and reduce the curing time.

Many repair projects require that repairs be opened to traffic within a few hours. To meet this challenge, a wide variety of rapid-setting and/or high-early-strength materials with a wide range of setting times are available, such as epoxy resin mortars and concretes (96, 98, 133). Many of these products are very sensitive to construction procedures or may be used only within very narrow temperature ranges. The manufacturer's directions regarding handling, mixing, placement, consolidation, screeding, and curing must be followed exactly and the durability of such materials in local climates must be carefully tested. These materials must also be thermally compatible with the concrete in the pavement. Differences in the coefficients of thermal expansion can cause repair failures.

Partial-depth repair failure is frequently caused by shrinkage separation from the edges of the repair, which weakens the repair and initiates progressive deterioration. Some agencies have successfully minimized these problems by using expansive mortars (e.g., high gypsum content mortars) for large repairs (98).

Bonding Agents

Sand/cement grouts have proven adequate when used with Portland cement concrete repair material, provided the repairs are protected from traffic for 24 to 72 hours. Portland cement grouts perform best when cured for at least 72 hours to allow adequate bond strength development, although lab tests conducted in Ohio indicate that bond strength develops sufficiently after 24 hours or less.

Epoxy bonding agents have been used successfully with both Portland cement concrete and proprietary repair materials to reduce required curing times to six hours or less.

Current Construction Practices

Location of Repair Boundaries

The extent of concrete deterioration may be greater than is visible at the surface. Weakened planes often develop in the slab with no visible deterioration of the surface. The extent of deterioration can be estimated by "sounding" the concrete with a solid steel rod, chains, or a ball peen hammer. Areas yielding a clear ringing sound are judged to be acceptable while those emitting a dull sound are considered weak. Sophisticated equipment to determine areas of deteriorated concrete is also commercially available (147, 148, 149).

All weak concrete must be located and removed if restoration is to be effective. Normally, the area marked for sawing is 3 to 4 in [76 to 102 mm] outside the defective area.

Removal of Deteriorated Concrete

The typical depth of concrete removal varies from 1 to 4 in [25 to 102 mm]. The removal method should provide an irregular surface to develop mechanical interlock between the repair material and the existing slab.

A vertical saw cut 1 to 2 in [25 to 51 mm] deep should be made beyond the boundary of the unsound area to be removed (see Figure 93). The cut boundary should be straight and vertical to provide a vertical face and clean-cut corners. Cutting repair boundaries with jackhammers produces "scalloped" boundaries that spall quickly. Some agencies successfully have used a 45 degree cut rather than a vertical cut (84).

The removal of unsound concrete is usually accomplished with jackhammers weighing up to 30 pounds [13.6 kg] maximum. Removal begins near the center of the area to be removed and proceeds toward (but not to) the edges. Care must be taken to avoid fracturing the sound concrete below the repair and undercutting or spalling repair boundaries. Removal in the areas of the repair boundaries must be completed with lighter (10- to 20-pound [4.5- to 9.1-kg]) hammers. Even hammers of this size fitted with gouge bits can damage sound concrete. Spade bits have been successfully used to remove unsound concrete without fracturing the underlying sound concrete. Large-area concrete removal can be facilitated by sawing shallow criss-cross or waffle patterns in the surface prior to using jackhammers, or by using pneumatic scarifiers, carbide-tipped cold milling machines or diamond blade grinding machines.

After removal, the bottom of the repair area is checked by "sounding" or other means to ensure that all deteriorated material has been removed. Remaining unsound concrete must be removed. If sound concrete cannot be reached (e.g., the area is unsound through the depth of the slab or unsound material cannot be removed because of reinforcing or load transfer devices) a full-depth repair is required. Small areas of full-depth repairs have been combined with partial-depth repairs, but these do not perform as well as regular full-depth repairs.

Cleaning the Repair Area

The surface must be cleaned following concrete removal. Any contamination of the surface will reduce the bond between the new material and the existing concrete. Dry sweeping, sandblasting and compressed air blasting are normally sufficient to provide a clean, irregular surface. Sandblasting is used to remove dirt, oil, and thin layers of unsound concrete, and is recommended for cleaning the surface. High-pressure water has also been used to remove contaminants, but sandblasting usually produces more reliable results. The compressed air must be free of oil, since contamination of the surface will prevent bonding. This can be checked by placing a rag over the air compressor nozzle and visually inspecting for oil.

With all cleaning methods, the prepared surface must be checked prior to placing the repair material. If the fingers pick up dust or other contaminants when rubbed across the prepared surface, the surface should be re-cleaned.

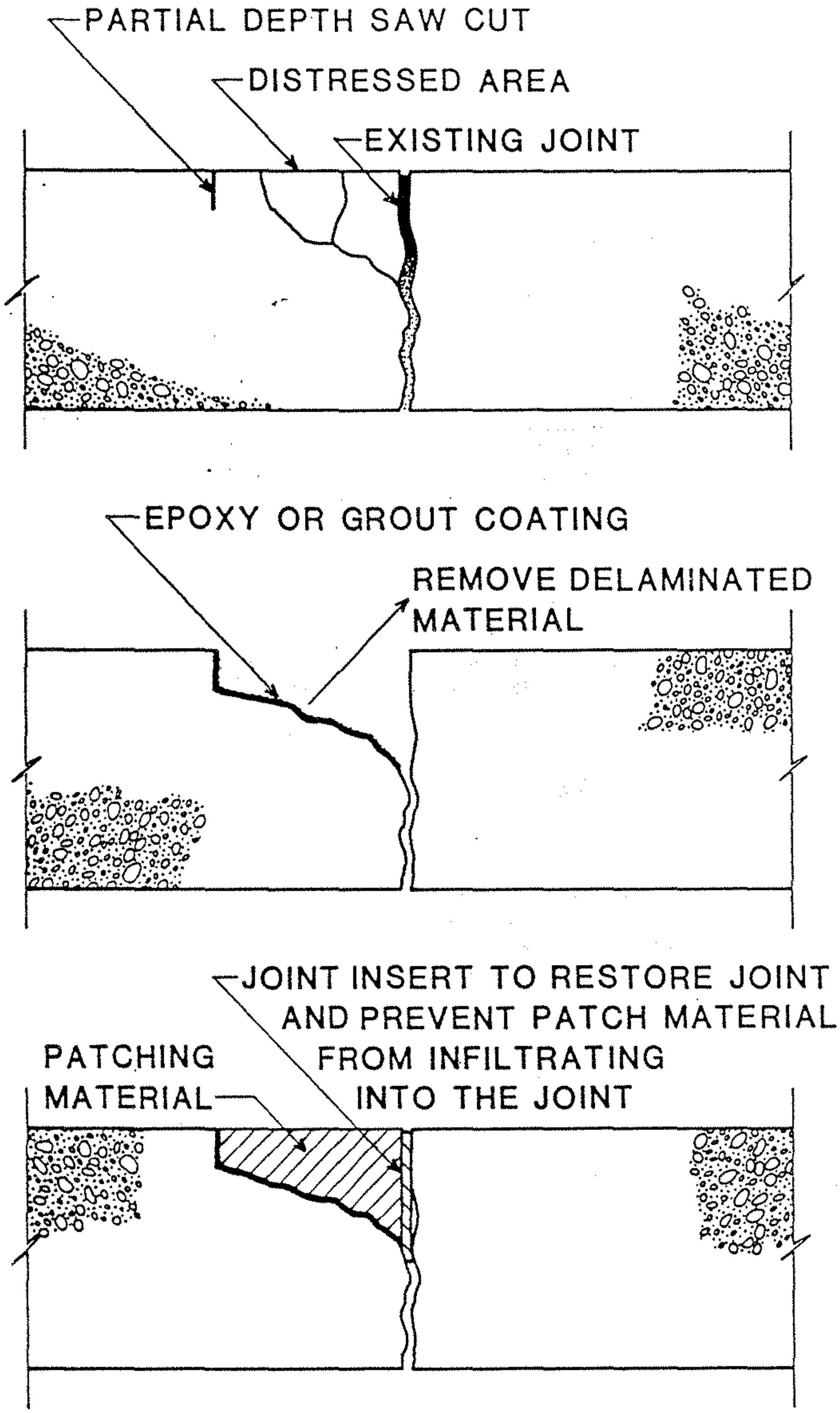


Figure 93. Illustration of Partial-Depth Repair Construction Procedures.

Joint Preparations

Partial-depth repairs placed adjacent to transverse, centerline, or shoulder joints require special construction preparations.

Partial-depth repairs placed directly in contact with an adjacent lane frequently spall due to curling stresses. This can be prevented by placing a thin bondbreaker material (e.g., a polyethylene strip) along the joint to separate the repair material from the adjacent material.

Partial-depth repairs placed directly against adjacent slabs (across the transverse joint) can be crushed by the compressive forces created when the slabs expand. This is the most frequent cause of partial-depth repair failure. This failure can be prevented by placing a strip of styrofoam or asphalt-impregnated fiberboard deep into the joint between the repair material and the adjacent slab (see Figure 93). This material must be able to absorb the joint movement. This material must also prevent intrusion of the repair material into the opening. Failure to do so can create compressive stresses at lower depths which can damage the repair. This material will also guard against damage due to deflection of the joint when the adjacent lane is trafficked during curing of the repair.

Some repairs have been successfully constructed without transverse joint forms by sawing the transverse joint to full depth as soon as the repair material has gained sufficient strength to permit sawing. However, any closure of the joint before sawing will fracture the repair. To avoid this problem, joints in partial-depth repairs placed across joints or cracks must be formed with compression-absorbing materials.

If the repair is to be placed adjacent to the shoulder, it must be formed along the shoulder. If the repair is allowed to flow into the shoulder, it may form a "key" which will restrict longitudinal movement of the slab and disrupt the repair.

All existing joint sealing or expansion joint materials should be removed to prevent contamination of the repair area. Sandblasting is an acceptable means of accomplishing this removal; solvents should never be used. Where spalling has been caused by a metal insert such as the Unitube, the insert should be removed along the entire length of the joint to avoid further spalls.

Application of Bonding Agent

After the surface of the existing concrete has been prepared, and just prior to placement of the repair material, it should be coated with a bonding agent to enhance bonding between the repair material and the existing concrete (see Figure 93). Common types of bonding agents include Portland cement/sand grouts and epoxy resins.

A saturated, surface-dry condition is desirable for application of cement grouts. When epoxies or other manufactured grouts are being used, the manufacturer's directions must be followed closely.

The grout should be placed immediately before the repair material is placed, before the grout can set. The bottom and all sides of the repair area should be thoroughly coated. This may be accomplished by brushing or "painting" the grout onto the concrete. Spray grouting may be appropriate for large repair areas.

Cement grout requires a minimum of 72 hours of curing prior to opening the area to traffic. Repairs that must be opened in less than 72 hours must use an epoxy bonding agent. Many epoxy bonding agents require 6 hours or less of curing prior to opening.

Consolidation of Materials

The repair material must be consolidated during placement. Consolidation removes trapped air from the fresh mix. Trapped air leads to poor repair durability, spalling, and rapid deterioration. For example, air voids located at the interface between the repair material and existing pavement can lead to total debonding and loss of repair material.

The internal vibrator and the vibrating screed give the most consistent results. The internal vibrator is often more readily available. Small repairs have been satisfactorily constructed with only hand tools; however, the repair material must be thoroughly worked with the tools to remove entrapped air.

Finishing

The repair surface is normally hand-trowelled to remove any remaining minor irregularities. Excess mortar from trowelling can be used to fill any saw cuts extending into the adjacent pavement at repair corners. Partial-depth repairs typically cover only a small percentage of the pavement surface and have little effect on skid resistance; however, the surface texture of the repair should match that of the surrounding slab as much as possible.

Curing Considerations

Curing is as important for partial-depth repairs as for full-depth repairs. Since partial-depth repairs often have large surface areas with respect to their volumes, moisture can be lost quickly. Inadequate attention to curing can lead to shrinkage cracks which contribute to premature repair failure.

All of the standard curing methods used for full-depth repair operations may be considered for partial-depth operations as well. The most effective curing procedure in hot weather is to apply a white-pigmented curing compound as soon as water has evaporated from the repair surface. This will reflect radiant heat while allowing the heat of hydration to escape and will provide protection for several days. Moist burlap and polyethylene can also be used, but they must be removed when the roadway is opened to traffic. In cold weather, insulating blankets or a tarp can be applied to provide more rapid curing and opening to traffic. Epoxies and proprietary repair materials should be cured as recommended by their manufacturers.

Performance of Partial-Depth Repairs

The performance of partial-depth repairs has been acceptable on projects where quality control was stringent. Studies on one heavily travelled toll road where several thousand partial-depth repairs had been installed showed that over 80 percent of the repairs were in excellent condition after five years of service (77). However, performance has often been unsatisfactory on projects with less stringent quality control. As many as 50 percent of the repairs have failed after two years of service on some projects. The most frequent causes of failure include loss of bond, compression failure (due to failure to reestablish the joint), variability of repair material, improper use of repair materials, insufficient consolidation, and incompatibility in thermal expansion between the repair material and the original slab.

Limitations and Other Considerations

Placement Temperature

Portland cement concrete repairs should not be placed when air or pavement temperatures are below 40°F [4°C]. At temperatures below 55°F [13°C] substantially longer curing times may be required, although the use of insulation and/or set accelerators will shorten curing times.

Costs

Repairing concrete pavement is generally very costly. When properly constructed at appropriate locations, partial-depth repairs can be more cost-effective than full-depth repairs (e.g., when replacing an entire joint to address small spalls). The cost of partial-depth repair depends on the size, number, and location of the repair areas, as well as the materials used. Lane closure time and traffic volume also affect production rates and costs.

Concurrent Work

On complete rehabilitation projects, spall repairs and partial-depth repairs should be completed after any undersealing and/or slab jacking, but prior to diamond grinding and joint sealing.

Precast Partial-Depth Repairs

Precast partial-depth repair has been considered as an alternative to conventional cast-in-place repairs. The feasibility of precast partial-depth repairs was demonstrated in New York in 1971 (36).

A study to evaluate mechanized approaches to partial-depth pavement repairs was initiated in Virginia in 1974 (133). Cast-in-place, hydraulically pressed, and wire fiber-reinforced precast repairs were installed in areas prepared using a special cutting machine. The precast slabs were 2 in [51 mm] thick and were seated and bonded using a sand-epoxy grout. Two years of performance data indicated that the hydraulically pressed precast repairs often fail because of voids in the epoxy bonding agent. This resulted in cracking under flexural loadings. The wire-reinforced repairs were better able to resist the formation of cracks or inhibited the propagation of cracks to the surface where they would be visible.

While the use of precast repairs was judged to provide satisfactory results in certain circumstances, production rates were generally low and costs relatively high. Precast partial-depth repairs will generally only be cost-effective when fairly short closure times are required for cast-in-place procedures.

CHAPTER IX

EVALUATION OF IN-SERVICE PARTIAL-DEPTH REPAIR INSTALLATIONS

AZ017199 - The pavement is a 9-in [229 mm] JPCP with contraction joints spaced every 15 feet [4.6 m]. It was constructed and opened to traffic in 1961. Partial-depth repairs and diamond grinding were performed in 1981 to repair joint spalling and reduce roughness.

The current annual traffic level for this project is approximately 1.5 million 18-kip [80-kN] ESALs in the outer lane, 1.0 million in the center lane, and 0.2 million in the inner lane. The cumulative traffic loading on the rehabilitation techniques since 1981 is approximately 4.5 million 18-kip [80-kN] ESALs in the outer lane, 3.5 million in the center lane, and 0.5 million in the inner lane. The cumulative traffic loading on the pavement since 1961 is approximately 11 million 18-kip [80-kN] ESALs in the outer lane, 9 million in the center lane, and 2 million in the inner lane.

Partial-depth repairs were placed at approximately 17% of the transverse joints. Repair boundaries were determined by sounding with a steel rod. The boundaries were cut to a minimum depth of 2.0 in [51 mm] using an air hammer, and the concrete was broken up with a 90-lb [41-kg] jackhammer. The repair area was then airblasted clean.

The repair material consisted of epoxy concrete. The latest time of day that repair installation was allowed was late afternoon. The transverse joints were maintained by the use of a joint insert. The repairs were cured with a membrane curing compound.

The partial-depth repairs are in excellent condition and faulting is minimal. However, some additional low-severity joint spalling has occurred, which suggests that the mechanism causing the spalling is still occurring. Since the transverse joint sealant was noted to be in very poor condition and incompressibles were observed in many of the joints, it appears that infiltration of incompressibles into the joints is the likely cause of the spalling.

The partial-depth repairs performed in 1981 were successful in repairing the existing spalling. However, immediate joint cleaning and resealing is required to prevent the occurrence of additional spalling.

IL280014/074005 - The pavements are 10-in [254 mm] JRCP with contraction joints spaced every 100 feet [30.5 m]. They were constructed in 1961 and opened to traffic in 1962. Full-depth repairs, partial-depth repairs, cement grout undersealing, diamond grinding, longitudinal subdrainage installation, and joint resealing were performed in 1984. Full-depth repairs were placed at 17% of the transverse joints in the outer lane and 48% of those in the inner lane to correct joint spalling and faulting. Localized spalls were repaired by partial-depth repairs at 20% of the transverse joints. Other rehabilitation techniques applied include cement grout undersealing, diamond grinding, longitudinal underdrains, joint resealing, and cracks sealing.

The current annual traffic level is approximately 1.0 million 18-kip [80-kN] ESALs in the outer lane and 0.2 million in the inner lane for I-280, and 0.6 million in the outer lane and 0.1 million in the inner lane for I-74. The cumulative traffic loading on the rehabilitation techniques since 1984 is 0.5 million ESALs in the outer lane and 0.25 million in the inner lane on I-280, and 0.25 million ESALs in the outer lane and 0.1 million in the inner lane on I-74. The cumulative traffic loading on the pavement since 1961 is 13.5 million ESALs in the outer lane and 2.6 million in the inner lane on I-280, and 10.5 million ESALs in the outer lane and 1.4 million in the inner lane on I-74.

Partial-depth repairs were placed at nearly 20% of the transverse joints. Repair boundaries (determined visually by the resident engineer) were cut with a diamond saw, and the concrete was removed with light jackhammer breakup. The repair area was then airblasted and brushed clean.

An epoxy grout was applied as the bonding agent for the partial-depth repairs. The repair material consisted of Portland cement concrete (Type I). The transverse joints were maintained by pouring the concrete against forms and later resealing. The concrete was consolidated with internal vibrators and vibrating screeds, and was finished by screeding, hand troweling, and brooming. The repairs were cured with burlap blankets, and opened to traffic 48 hours after placement.

All of the existing partial-depth repairs are in excellent condition and are performing well. However, approximately 47% of the regular contraction joints in the outer lane and 40% of those in the inner lane exhibited medium-severity spalling. This suggests that either an insufficient number of partial-depth repairs were placed in 1984, or the mechanism responsible for the spalling was not corrected by the 1984 work and has caused significant joint distress to develop since that time. Since the transverse joint sealant was noted to be in very poor condition (adhesion failure was observed at 89% of the joints) and incompressibles were observed in many of the joints, it appears that infiltration of incompressibles into the joints is the likely cause of the spalling.

The partial-depth repairs performed in 1984 were successful in repairing the existing spalling at the joints where they were placed. However, a significant amount of joint spalling was not repaired in 1984. Additional partial-depth repairs are required to repair existing joint spalling. Joint cleaning and resealing is also required to prevent the occurrence of further spalling in the future.

LA010151 - This pavement is a 10-in [254 mm] JRCP with a joint spacing of 58.5 feet [17.8 m]. It was constructed and opened to traffic in 1971. This project was rehabilitated in 1984 as part of Federal Highway Administration Demonstration Project No. 69, "Portland Cement Concrete Pavement Restoration" (24). The rehabilitation techniques applied to the pavement were full-depth repairs, cement grout undersealing, load transfer restoration, diamond grinding, joint resealing, crack repair and longitudinal underdrains.

The current annual traffic level for this project is approximately 1.4 million 18-kip [80-kN] ESALs in the outer lane and 0.4 million in the inner lane. The cumulative traffic loading on the rehabilitation techniques since 1984 is approximately 2.8 million ESALs in the outer lane and 0.8 million in the inner lane. The cumulative traffic loading on the pavement since 1971 is approximately 10 million ESALs in the outer lane and 2 million in the inner lane.

Partial-depth repair boundaries were determined visually by the resident engineer. The boundaries were cut with a diamond saw to an average depth of 4.0 in [102 mm], and the concrete was removed with light jackhammer breakup. The repair area was then brushed clean.

A cement grout was applied as the bonding agent for the partial-depth repairs. The repair material consisted of Portland cement concrete (Type I) with calcium chloride accelerator added. The transverse joints were maintained by pouring the concrete against forms and later resealing with silicone. The concrete was finished by hand troweling. No curing compound was used. The repairs were opened to traffic 24 hours after placement.

The partial-depth repairs were placed at 60% of the transverse joints to correct localized spalling, primarily at the intersections of the transverse joints and the longitudinal centerline joint. A crack survey conducted by the Louisiana Department of Transportation and Development [LDOTD] prior to the restoration work showed that this type of spalling had occurred at 97% of the transverse joints. LDOTD personnel attributed this spalling to improper forming of the centerline joint in the vicinity of the transverse joints (24).

Loss of patch material was noted on 38% of the partial-depth repairs in a 1985 survey. Typically this material loss was in the range of 10% to 20% of the material, but went as high as 40% at a few repairs. Most of the repairs which exhibited material loss were located in the outer lane. The nature of this material loss was typically cracking and crumbling of the concrete within the patch boundaries (see Figure 94).

Longitudinal cracks and cracks at the pavement slabs' outer corners were observed at several joints during the 1984 (LDOTD) and the 1985 (ERES) surveys. LDOTD personnel attributed this cracking largely to dowel bar misalignment (24). Longitudinal centerline cracking at any locations other than the transverse joints was not observed during either survey. It may be, that the longitudinal joint spalling which occurred at the transverse joints was not due to poor longitudinal joint construction, but rather to the same mechanisms which caused the longitudinal and corner cracking at the transverse joints (i.e., dowel bar misalignment or compressive stress buildup). If this is true, the corner spalls may have been at the top two or three inches [51 to 76 mm] of cracks through the full slab thickness. This may explain why partial-depth repairs which exhibited material loss were for the most part located in the outer lane, where most of the longitudinal and corner cracking at transverse joints occurred.

The poor performance of the partial-depth repairs in the outer lane after only one year of service suggests that they were inappropriate for this project. Replacement of those partial-depth repairs which have experienced substantial material loss with full-depth repairs should be considered in conjunction with any future rehabilitation work done to repair the transverse joints in the outer lane with longitudinal and corner cracking.

Figure 94. Photo of Partial-Depth Repair on LA010151.

NE080279 - The pavements is a 9-in [254 mm] JRCP with contraction joints spaced every 46.5 feet [14.2 m]. It was constructed in 1962 and opened to traffic in 1963. Full-depth repairs, partial-depth repairs, pressure relief joint installation, and joint resealing were performed in 1982. Full-depth repairs were placed at 10% of the transverse joints in the outer lane and 4% of those in the inner lane, as well as at major cracks, to correct spalling caused by expansion of reactive aggregate. Localized spalls at joints and major cracks, ostensibly due to the same cause, were repaired by partial-depth repairs. Pressure relief joints were installed at 2000-ft [610-m] intervals to reduce pressure buildup in the pavement caused by the reactive aggregate expansion. Joint resealing was performed at all original contraction joints.

The current annual traffic level for this project is approximately 1.3 million 18-kip [80-kN] ESALs in the outer lane and 0.3 million in the inner lane. The cumulative traffic loading on the rehabilitation techniques since 1982 is approximately 4.0 million 18-kip [80-kN] ESALs in the outer lane and 0.9 million in the inner lane. The cumulative traffic loading on the pavement since 1963 is approximately 14 million 18-kip [80-kN] ESALs in the outer lane and 3 million in the inner lane.

The partial-depth repair boundaries were determined visually by the resident engineer. The boundaries were cut with a diamond saw to an average depth of 2.0 in [51 mm], and the concrete was broken up with a jackhammer and removed.

Epoxy concrete was used as the partial-depth repair material. An epoxy primer was used as a bonding agent. The repairs were placed by early afternoon, finished by hand troweling, and allowed to cure a minimum of 4 hours. A bondbreaker was used to separate the repair material from existing joints.

In general the partial-depth repairs are not performing well. Some of the partial-depth repairs were placed at working cracks without the crack being reestablished, and as a result, have experienced spalling and material loss (typically 5 to 15 percent of the repair area). The State of Nebraska has discontinued use of epoxy concrete for partial-depth repairs because of its high costs and poor performance.

Full-depth repairs would probably have been more appropriate for working cracks on this project (see Figure 95). Replacement of the partial-depth repairs with full-depth repairs could be performed in conjunction with other rehabilitation work which is required on this pavement in the near future.

NE080382 - The pavements is a 9-in [254 mm] JRCP with contraction joints spaced every 46.5 feet [14.2 m]. It was constructed and opened to traffic in 1962. Full-depth repairs, partial-depth repairs, pressure relief joint installation, and joint resealing were performed in 1982. Full-depth repairs were placed at 13% of the transverse joints in the outer lane and 9% of those in the inner lane, as well as at major cracks, to correct spalling resulting from the use of a mildly reactive aggregate. Localized spalls at joints and major cracks were repaired by partial-depth repairs constructed with epoxy cement. Pressure relief joints were

Figure 95. Photo of Partial-Depth Repair at Major Crack on NE080279.

installed at 1-mile [1.6-km] intervals to reduce pressure buildup in the pavement caused by the reactive aggregate expansion. Joint resealing was performed at all original contraction joints.

The current annual traffic level for this project is approximately 1.5 million 18-kip [80-kN] ESALs in the outer lane and 0.5 million in the inner lane. The cumulative traffic loading on the rehabilitation techniques since 1982 is approximately 5.0 million 18-kip [80-kN] ESALs in the outer lane and 1.3 million in the inner lane. The cumulative traffic loading on the pavement since 1963 is approximately 19.5 million 18-kip [80-kN] ESALs in the outer lane and 5.3 million in the inner lane.

Boundaries for the partial-depth repairs were determined visually by the resident engineer and cut with a diamond saw to an average depth of 2.0 in [51 mm]. The deteriorated concrete was broken up with a jackhammer and removed.

Epoxy concrete was used as the partial-depth repair material. An epoxy primer was used as a bonding agent. The repairs were placed by early afternoon, finished by hand troweling, and allowed to cure a minimum of 4 hours. A bondbreaker was used to separate the repair material from existing joints.

Five partial-depth repairs were found in the surveyed 2050 feet [625 m] of this project. Three of these were placed at the centerline longitudinal joint at mid-slab. The other two were placed adjacent to a full-depth repair. The partial-depth repairs are in good condition. Some minor material loss was noted during the 1985 survey, but no scaling, debonding, cracking, or other distress was noted on the partial-depth repairs. After three years of service, the epoxy cement partial-depth repairs placed on this project appear to be performing well.

NE080404 - The pavements is a 10-in [254 mm] JPCP with contraction joints spaced every 16.3 feet [5 m]. It was constructed and opened to traffic in 1960. Full-depth repairs, partial-depth repairs, and joint resealing were performed in 1984. The full-depth repairs were primarily complete replacements of cracked slabs, placed at approximately 3% of the slabs. Partial-depth repairs were placed to correct localized spalling at transverse and longitudinal joints.

The current annual traffic level for this project is approximately 1.2 million 18-kip [80-kN] ESALs in the outer lane and 0.4 million in the inner lane. The cumulative traffic loading on the rehabilitation techniques since 1984 is approximately 1.2 million 18-kip [80-kN] ESALs in the outer lane and 0.4 million in the inner lane. The cumulative traffic loading on the pavement since 1963 is approximately 15 million 18-kip [80-kN] ESALs in the outer lane and 4 million in the inner lane.

Boundaries for the partial-depth repairs were determined visually by the resident engineer and cut with a diamond saw to an average depth of 2.0 in [51 mm]. The concrete was broken up with a jackhammer and removed.

Epoxy concrete was used as the partial-depth repair material. An epoxy primer was used as a bonding agent. The repairs were placed by early afternoon, finished by hand troweling, and allowed to cure a minimum of 4 hours. A bondbreaker was used to separate the repair material from existing joints.

In general, the partial-depth repairs are not performing well (see Figures 96 and 97). Some of the partial-depth repairs were placed at working cracks without the crack being reestablished, and as a result, have experienced spalling and material loss (as much as 25% of the repair area). Full-depth repairs would probably have been more appropriate for working cracks. Partial-depth repairs which were not placed at working cracks are performing well. This could be corrected by replacement of the partial-depth repairs with full-depth repairs of the working cracks.

OH077053 - The pavement is a 9-in [229 mm] JRCP with contraction joints spaced every 60 feet [18.3 m]. It was constructed and opened to traffic in 1967. CPR work was performed on this pavement in 1982 as a field study of the techniques developed in NCHRP 1-21, specifically full-depth repairs, partial-depth repairs, undersealing, load transfer restoration, diamond grinding, and joint resealing (37). All rehabilitation techniques were performed in the outer lane only. It was noted that construction practices on this demonstration project were generally poor.

The current annual traffic level for this project is approximately 0.6 million 18-kip [80-kN] ESALs in the outer lane and 0.1 million in the inner lane. The cumulative traffic loading on the rehabilitation techniques since 1982 is approximately 2.1 million ESALs in the outer lane and 0.5 million in the inner lane. The cumulative traffic loading on the pavement since 1967 is approximately 7 million ESALs in the outer lane and 1 million in the inner lane.

The partial-depth repair boundaries were determined visually by the resident engineer and cut with a diamond saw to an average depth of 2.0 in [51 mm]. The concrete was broken up with a light jackhammer and removed.

Two materials were used for the partial-depth repairs on this project: Portland cement concrete (bonded to the existing concrete with a cement grout) and polymer concrete (bonded with a commercial primer, Silikal). There were no time or temperature restrictions on placement of the repairs. The concrete was finished by hand troweling and cured with a curing compound. Transverse joints were then reestablished by sawing.

Three partial-depth repairs were present within the surveyed portion of this project. Two of these are in good condition and have no distresses. The third repair exhibits some material loss. It is not known which of the two material types used on this project (Portland cement concrete and polymer concrete) were used on these three repairs.

The project has experienced some distress such as transverse joint and corner spalling, joint faulting, longitudinal and transverse cracking and deteriorated sealant condition. Where spalling was observed, it was of low severity. Longitudinal joint spalling was not observed in the repairs. Transverse cracks were observed extending across both lanes of the pavement at approximately 15-20 ft [4-6 m] intervals. The contraction joint sealant

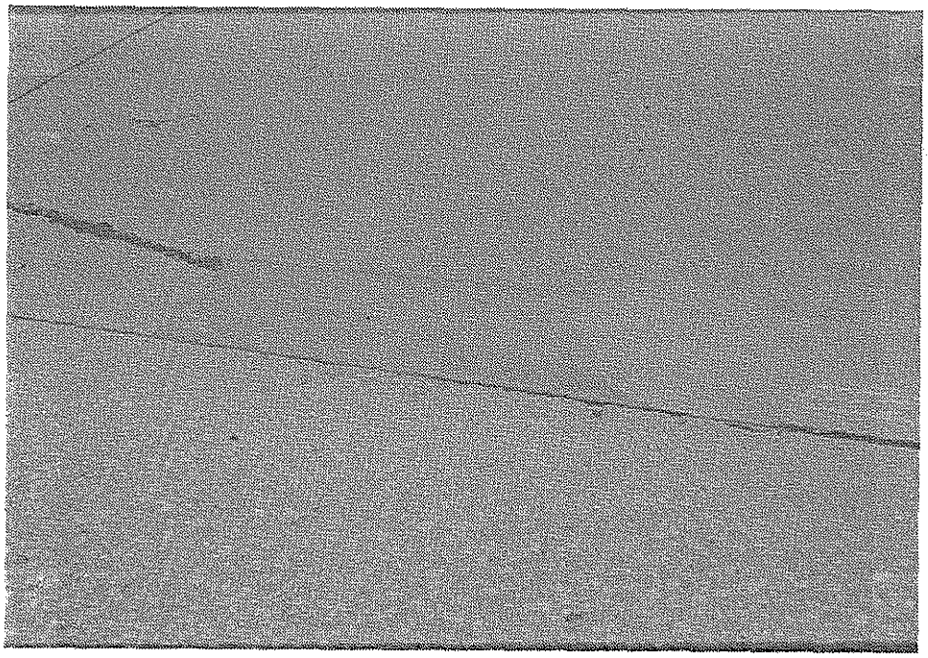


Figure 96. Photo of Partial-Depth Repair at Working Crack on NE080404.

Figure 97. Photo of Material Loss of Partial-Depth Repair on NE080404.

was absent in some of the original inner lane contraction joints, and varying amounts of incompressibles were present in both lanes. The transverse joint spalling and corner spalling were slightly worse in the inner, unrepaired lane.

It is difficult to assess the success of the partial-depth repairs on this project due to the small number of repairs surveyed and their inconsistent condition. However, further joint spalling and infiltration of incompressibles has occurred since the time the CPR work was done. Full-depth repairs are scheduled for this project in 1987.

VA044000 - Virginia Route 44 was built in 1967 as a 9-inch [229 mm] JRCP with a transverse joint spacing of 61.5 feet [18.7 m]. The project has been rehabilitated with full-depth repairs in 1984, partial-depth repairs and pressure relief joints in 1976 and 1984, and joint resealing in 1973, 1976 and 1984.

The current annual traffic level for this project is approximately 0.4 million 18-kip [80-kN] ESALs in the outer lane and 0.2 million in the inner lane. The cumulative traffic loading on the rehabilitation techniques applied in 1976 is approximately 2.5 million ESALs in the outer lane and 0.9 million in the inner lane, and 0.5 million in the outer lane and 0.1 million in the inner lane for those applied in 1984. The cumulative traffic loading on the pavement since 1967 is approximately 4 million ESALs in the outer lane and 1 million in the inner lane.

The full- and partial-depth repairs were placed to address joint spalling caused by the Unitube inserts used to form the transverse joints. The partial-depth repairs placed in 1976 used calcium aluminate cement concrete for high early strength. The 1984 partial-depth repairs used Type III Portland cement concrete.

The partial-depth repairs were constructed by essentially the same techniques in 1976 and 1984. Repair boundaries were determined visually by the resident engineer and cut to an average depth of 2.0 in [51 mm] with a diamond saw. The deteriorated concrete was broken up with a light jackhammer and removed.

The 1976 (calcium aluminate cement concrete) repairs were bonded to the existing concrete with a calcium aluminate cement slurry. The concrete was finished by hand troweling and cured with a membrane curing compound. Repair placement was restricted to times when the air temperature was between 45°F [7°C] and 80°F [27°C].

The 1984 (Type III Portland cement concrete) repairs were bonded to the existing concrete with a cement slurry. They were finished and cured in the same manner as the full-depth repairs placed on this project concurrently.

Many of the 1976 repairs are cracked and exhibit scaling or material loss (typically 5% to 10%). Also, spalling of the concrete slab was often noted adjacent to the 1976 repairs. Nine of the forty-two surveyed 1976 repairs (21%) exhibited significant distress. The 1984 repairs are in good condition. They exhibit no scaling or material loss, but one of them has cracked.

The poor condition of the 1976 partial-depth repairs is likely due not so much to their age as to the calcium aluminate cement present. Research has found calcium aluminate cement to be subject to a chemical conversion at temperatures above approximately 77°F [25°C] which can cause substantial strength loss. This can occur at any time in the cement's service life, but is especially detrimental during initial curing. It is likely, therefore, that the distresses noted for the 1976 partial-depth repairs on this project did not develop slowly over nine years' time, but instead were the result of substantial strength loss in the repair material within the first summer of service. This is consistent with reports of poor performance of full- and partial-depth repairs constructed with calcium aluminate cement on several projects in Virginia. Calcium aluminate cement is not prohibited for partial-depth repairs in Virginia at this time.

The success of the 1984 partial-depth repairs is difficult to assess on the basis of only one year of service. However, better performance is expected from these repairs than that given by the 1976 repairs, because of the use of conventional high-early-strength concrete. Replacement of the 1976 partial-depth repairs with new partial-depth repairs is warranted. Also, survey results show that most of the original contraction joints exhibited low-severity transverse joint spalling, with a few displaying medium-severity spalling. Incompressibles were present in many of the transverse joints. Whether the spalling is due to the presence of Unitubes or infiltration of incompressibles is unknown. In either case, further partial-depth repair work and joint resealing are needed to correct existing spalling and prevent its recurrence.

VA064202 - This pavement was constructed in 1966 as a 9-inch [229 mm] JRCP with a transverse joint spacing of 61.5 feet [18.7 m]. The rehabilitation methods applied to this pavement were full-depth repairs and partial-depth repairs in 1984 and 1976, pressure relief joint installation in 1982, and joint resealing in 1984 and 1976.

The full- and partial-depth repairs were placed to address joint spalling caused by the Unitube inserts used to form the transverse joints. The partial-depth repairs placed in 1976 used calcium aluminate cement concrete for high early strength. The 1984 partial-depth repairs used Type III Portland cement concrete.

The current annual traffic level for this project is approximately 0.4 million 18-kip [80-kN] ESALs in the outer lane and 0.1 million in the inner lane. The cumulative traffic loading on the 1976 rehabilitation techniques is approximately 2 million ESALs in the outer lane and 0.6 million in the inner lane, and approximately 0.4 million in the outer lane and 0.1 million in the inner lane on the 1984 repairs. The cumulative traffic loading on the pavement since 1966 is approximately 4 million ESALs in the outer lane and 1 million in the inner lane.

The partial-depth repairs were constructed by essentially the same techniques in 1976 and 1984. Repair boundaries were determined visually by the resident engineer and cut to an average depth of 2.0 in [51 mm] with a diamond saw. The deteriorated concrete was broken up with a light jackhammer and removed.

The 1976 (calcium aluminate cement concrete) repairs were bonded to the existing concrete with a calcium aluminate cement slurry. The concrete was finished by hand troweling and cured with a membrane curing compound. Repair placement was restricted to times when the air temperature was between 45°F [7°C] and 80°F [27°C].

The 1984 (Type III Portland cement concrete) repairs were bonded to the existing concrete with a cement slurry. They were finished and cured in the same manner as the full-depth repairs placed on this project concurrently.

Some of the 1976 repairs (calcium aluminate cement) exhibit scaling and material loss (typically 5% to 10%). The 1984 repairs are in good condition. They exhibit no scaling or material loss, but one of them has cracked.

As with the partial-depth repairs performed on Virginia Route 44, the poor condition of the 1976 partial-depth repairs is likely due not so much to their age as to the calcium aluminate cement present. However, the calcium aluminate cement partial-depth repairs on I-64 have performed better than those on Route 44, even though they were placed in the same year and the temperature conditions throughout the year are essentially the same at the two locations.

The success of the 1984 partial-depth repairs is difficult to assess on the basis of only one year of service. However, better performance is expected from these repairs than that given by the 1976 repairs, because of the use of conventional high-early-strength concrete. The deterioration of the 1976 repairs is sufficiently minor that replacement is not warranted.

VA081147 - This pavement was constructed in 1965 as a 9-in [229 mm] JRCP with a transverse contraction joint spacing of 61.5 feet [18.7 m]. Rehabilitation techniques performed in 1972, 1976, 1982, and 1984 include full-depth repairs, partial-depth repairs, cement-grout undersealing, diamond grinding, longitudinal underdrains, load transfer restoration, joint resealing, and pressure relief joint installation.

The partial-depth repairs were placed to address joint spalling caused by the Unitube metal joint-forming inserts. Some surface deterioration due to high steel was also corrected with partial-depth repairs. The only partial-depth repairs surveyed were those placed in 1984.

The current annual traffic level for this project is approximately 0.8 million 18-kip [80-kN] ESALs in the outer lane and 0.2 million in the inner lane. The cumulative traffic loading on the partial-depth repairs placed in 1984 is approximately 0.8 million ESALs in the outer lane and 0.2 million in the inner lane. The cumulative traffic loading on the pavement since 1965 is approximately 8 million ESALs in the outer lane and 1 million in the inner lane.

Boundaries for the partial-depth repair were determined visually by the project engineers and cut to a minimum depth of 2.0 in [51 mm] with a diamond saw. A light jackhammer was used to break up the concrete.

Type III Portland cement was used for the 1984 repairs. A cement slurry was used to bond the repairs to the existing concrete. The concrete was finished by hand troweling. The repairs were allowed to cure for a minimum of 6 hours before being opened to traffic. Placement of the repairs was restricted to times when the air temperature was between 45°F [7°C] and 80°F [27°C].

The surveyed partial-depth repairs are in good condition, with no scaling, debonding, cracking, or material loss. Their long-term performance is difficult to predict, however, on the basis of one year of service. No information is available about the performance of older partial-depth repairs on this project.

VA095000 - This pavement was built in 1963 as a 9-inch [229 mm] JPCP with a contraction joint spacing of 20 feet [6.1 m]. It was rehabilitated with partial-depth repairs in 1968, 1983, and 1984 and full-depth repairs and joint resealing in 1984. The partial- and full-depth repairs were placed at nearly all of the transverse joints in the outer lane and at approximately 40% of the joints in the inner lane, to correct spalling caused by Unitube joint forming inserts.

The current annual traffic level for this project is approximately 0.6 million 18-kip [80-kN] ESALs in the outer lane and 0.1 million in the inner lane. The cumulative traffic loading on the partial-depth repairs placed in 1983 is approximately 1.2 million in the outer lane and 0.2 million in the inner lane, and is approximately 0.6 million ESALs in the outer lane and 0.1 million in the inner lane on the repairs placed in 1984. The cumulative traffic loading on the pavement since 1963 is approximately 7.5 million ESALs in the outer lane and 1 million in the inner lane.

Boundaries for the partial-depth repairs were determined visually by the resident engineer and sawed to between 3.0 in [76 mm] and 4.0 in [102 mm] with a diamond saw. A minimum width of 12 in [305 mm] was required for the repairs. The deteriorated concrete was broken up with a light jackhammer and removed.

All of the partial-depth repairs were constructed with Type III Portland cement concrete. A cement slurry was used to bond the repairs to the existing concrete. The repairs were finished by hand troweling and cured with a membrane curing compound. Placement was not permitted when the air temperature was below 55°F [13°C].

All of the partial-depth repairs are in good condition, with no scaling, debonding, cracking, or material loss (see Figures 98 and 99). The full-depth repairs, however, are experiencing significant cracking, faulting, and pumping. Of the two rehabilitation techniques, partial-depth repair is believed to have been the more successful and more appropriate for this project.

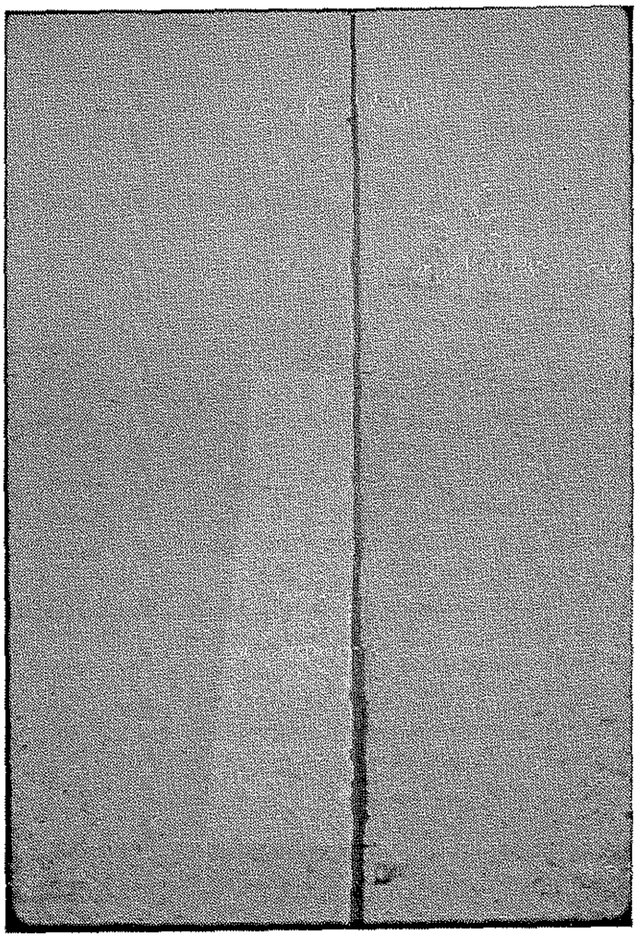


Figure 98. Photo of Partial-Depth Repair on VA095000.

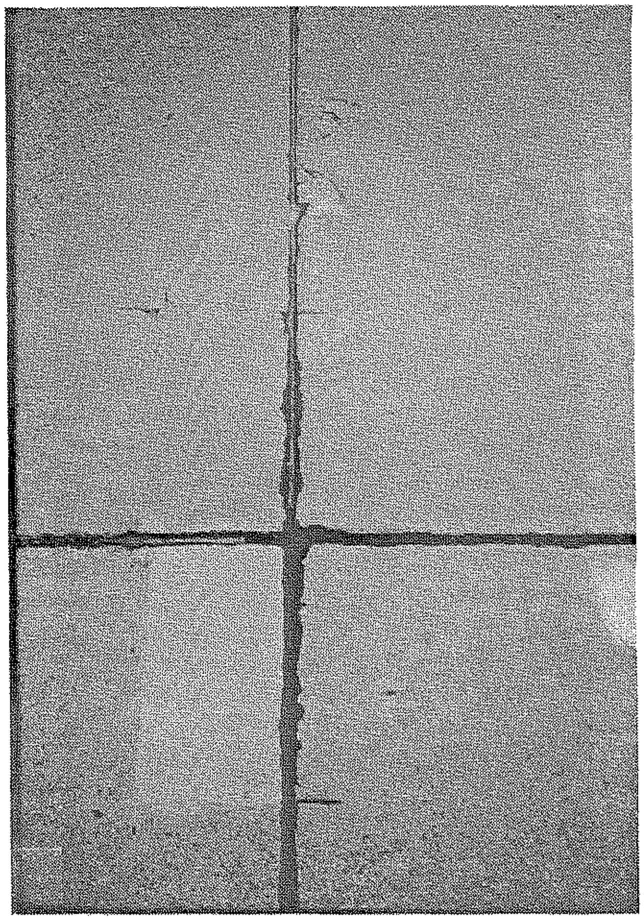


Figure 99. Photo of Partial-Depth Repair on VA095000.

CHAPTER X

PANEL CONCLUSIONS AND RECOMMENDATIONS

PERTAINING TO THE DESIGN AND USE OF PARTIAL-DEPTH REPAIRS

Conclusions

1. Spalls are typically caused by infiltration of incompressibles, Unitube joint forming inserts, low- to medium-severity "D" cracking or expansion of reactive aggregate, inadequate cover of reinforcing steel, or surface scaling.

2. Partial-depth repairs have proven successful and cost-effective for addressing localized spalls at transverse or longitudinal joints where the spall does not extend through more than one half of the depth of the slab nor below the depth of steel reinforcing or dowels.

3. Partial-depth repairs are not suitable for cracks which extend through more than half of the slab thickness, such as might be caused by compression cracking, transverse or longitudinal cracking resulting from late joint sawing or inadequate joint sawcut depth, or transverse or longitudinal shrinkage or fatigue cracking.

4. Materials that have been used successfully for partial-depth repairs include Type III Portland cement concrete with or without an accelerating admixture, proprietary rapid-setting materials, and epoxy concrete. Partial-depth repairs constructed using calcium aluminate cement concrete have performed poorly because they have experienced large amounts of shrinkage and debonding.

5. Success of partial-depth repairs is often dependent upon the quality of construction and/or the contractor's experience with partial-depth repairs.

Recommendations

1. Coring of representative joints on the project is recommended to determined the depth of spalling and assess the suitability of partial-depth repairs.

2. Partial-depth repairs placed across working joints or cracks must include reestablishment of the joint or crack directly above the existing joint or crack in the underlying slab.

3. Calcium aluminate cement concrete should not be used for partial-depth repairs.

4. Where existing joint sealant conditions are poor and incompressibles are observed to have infiltrated into the joints, cleaning and resealing of all joints and cracks on the project should be performed concurrently with partial-depth repair.

CHAPTER XI

JOINT RESEALING BACKGROUND INFORMATION

Joint Resealing

Introduction

Resealing concrete pavement joints is an important phase of restoration. Field surveys indicate that only a small portion of joints and cracks are adequately sealed (27). Joint resealing is necessary when incompressibles can enter the joint and/or water can enter the pavement structure. The damage resulting from this infiltration is well documented (13, 20, 62, 70, 73, 93, 137). Excess water can accelerate damage due to base erosion and loss of support, and incompressibles can cause joint deterioration and blowups.

Pavement life may be extended by sealing joints and cracks. The benefits of resealing include the removal of incompressibles, the prevention of further intrusion, and the reduction of water infiltration. Historically these benefits have been short-lived due to relatively rapid joint sealant deterioration. Joint sealants typically become ineffective within one to four years after installation. Some recently developed sealant materials can extend the effective life to seven or more years when properly installed.

Appropriate Uses and Locations

Resealing can be effective in reducing the infiltration of water through joints and intrusion of incompressibles into joints in most climatic regions. Joint resealing may even be cost-effective where only water *or* incompressibles are a problem.

The cost-effectiveness of resealing depends on the condition of existing sealant, presence of incompressible materials, climate, traffic volumes, the quality of surface and subbase paving materials, and the cost of resealing versus the cost of repairing the damage that will result without resealing. If a drainage survey indicates that moisture in the pavement structure will, or has, accelerated distress, and that the joints are the source of that water, then joint and crack resealing is essential.

Resealing may *not* be cost-effective on roads that carry low volumes of trucks, especially in dry climates. *The extent of distress caused or accelerated by free moisture and incompressibles in the pavement structure must be considered in deciding whether or not to seal joints or cracks*.

All major sources of surface water infiltration must be sealed to reduce water infiltration. These sources include the transverse joints, longitudinal lane/shoulder joints, longitudinal joints between traffic lanes, and cracks in the pavement surface as well. The lane/shoulder joint is normally the largest source of water entering most pavement structures (7). Adequately designed and constructed longitudinal underdrains will reduce the amount of water remaining in the structure; however, the longitudinal lane/shoulder joint may still need to be sealed, since large amounts of water passing through the joint may carry fines through the drainage system and create edge voids.

Construction joints and longitudinal joints between traffic lanes generally experience little deformation, and sealing them may not be as critical as contraction and lane/shoulder joints. However, in multi-lane pavements with a constant cross slope, the longitudinal joints can provide a major route for water to enter the pavement structure.

Normally, only the transverse joints need to be sealed to prevent pressure damage. Cleaning incompressibles from these joints during the resealing procedure will also produce significant benefits.

Sealant Materials

Properties

Many sealants are available, and many have very different properties that affect their field performance. These properties include durability, extensibility, cohesiveness, and adhesiveness (12, 13, 14, 25, 40, 70, 93, 106, 128). _All sealants considered for use must be thoroughly tested by the user agencies prior to installation to increase the probability of good field performance_.

A nondurable sealant will blister, harden and crack in a relatively short time. These reactions reduce the effectiveness of the seal.

Extensibility is the ability of the sealant to deform without rupturing. The more extensible the sealant, the further the joint may open without damaging the sealant. Soft, low-modulus sealants are very extensible, but they must also be stiff enough to resist the intrusion of incompressibles. In addition, the sealant must retain its extensibility over a wide temperature range since the maximum joint or crack opening normally occurs during the coldest seasons.

Cohesiveness is the internal resistance of the sealant to tensile failure. Many sealants harden over time and become more susceptible to tensile failure.

Adhesive failure is the separation of the sealant from the sidewall. The bonding of the sealant must be maintained over its life.

Types of Sealants

Categories of sealants currently used include:

- Field-Poured Sealants (Self-levelling)
  - Hot-Poured
  - Cold-Poured
- Field-Poured Sealants (Non-self-levelling)
- Preformed Compression Seals

The use of high-quality sealants is recommended for long-lasting performance. Sealant materials currently reported as giving fair-to-good performance include improved rubber asphalt, low-modulus silicone, polyvinyl chloride, polyurethane, and preformed compression sealants. Each of these materials has different durability, extensibility and bonding properties, and cost. The properties of several types of sealants are discussed below.

Field-poured self-levelling liquid sealants include many hard paving asphalts and air-blown asphalts that are mixed with mineral fillers, rubber asphalts and other rubber products. These materials are generally poured hot and stiffen upon cooling.

Most field-poured liquid sealants on the market today are designed to withstand strains of roughly 25 percent of their original width. Thus, a 0.5-in [13 mm] wide joint can open no more than 0.125 in [3 mm] before the strain in the sealant exceeds the 25 percent limit. This 25 percent strain limit defines the working range of most field-poured liquid sealants. The working range of the sealant must be greater than the working range of the joint (as determined by estimating mean horizontal joint movements or measuring typical joint movements on previously constructed projects of similar design).

The width of the reservoir must be adjusted to achieve a satisfactory depth-to-width ratio (or shape factor). The depth of the sealant in the reservoir must meet the manufacturer's recommendations to ensure sufficient bonding area on the reservoir walls.

Field-poured liquid sealants used in original sealing are used for resealing operations also. AASHTO M173, ASTM D1190, ASTM 3405 and Federal Specification SS-S-1401B include the hot-applied thermoplastic asphalt-rubber sealant compounds. These sealants provide the shortest life but are the cheapest.

ASTM D1850 includes cold-applied mastic, single or multiple application types. Improved rubber/asphalt formulations that meet state specifications provide a somewhat better life.

Many sealants meeting these specifications have been ineffective in resealing operations. Bond and cohesion failures occur in cold weather and excessive extrusion of the sealant occurs during the summer. Maintenance resealing is often necessary to replace sealants lost to extrusion or removal by snowplows. Many of these materials provide poor resistance to the intrusion of incompressibles, although acceptable results have been obtained using the improved rubberized asphalt products.

Sealants meeting ASTM D3406 specifications are hot-poured elastomeric polyvinyl chloride (PVC) materials which have performed for several years in some installations. These sealants use special chemicals which combine with the base materials when heated to a specified temperature and polymerize. The recommended shape factor (ratio of depth to width) for hot-poured sealants is between 1 and 2.

The newer low-modulus silicone sealants have superior bonding potential, durability, and cohesion which allow them to be placed thinner than the other sealants. These sealants are usually placed with shape factors of about 0.5; however, a minimum thickness of 0.25 in [6 mm] is recommended to achieve bonding with the joint sidewall. Silicone sealants are not self-levelling. They must be tooled to produce a uniform surface, force the sealant into contact with the sidewall, and produce the correct sealant shape. After tooling, the sealant must be cured to promote strength gains. Silicone sealants have performed adequately for over seven years in the first installations (144).

Cold-applied, two-component sealants (e.g., polymer sealants) use two components that are mixed as the material is placed in the joint. These sealants require a special application nozzle and careful control of the application equipment. A curing period is required for the material to gain strength. The behavior and performance of polymer sealants may be comparable to that of silicone sealants, except that in some cases polymer sealants are more difficult to use because of their two-component application.

While liquid sealants are designed to withstand both tension and compression, preformed compression seals must remain in compression for their entire life. There is little or no bond between the compression seal and the sidewalls of the joint to sustain tension. Performance depends largely on the ability to maintain sufficient contact with the joint walls. Compression seals are designed to compress through an internal web structure, as depicted in Figure 100. This webbing provides continual thrust against the joint sidewalls to hold the seal in place and resist the infiltration of water and incompressibles. These seals must always be compressed to <u>at least</u> 80 percent of their uncompressed width (e.g., a 1-in [25-mm] wide seal must always be compressed to 0.8 in [20 mm] wide in the joint). If compression is lost, the seal will be free in the joint and will likely come out of the joint.

Maximum compression should be limited to 50 percent of uncompressed seal width. Seals subjected to greater compressions for extended periods may take a compression "set." The webs will bond to each other, the seal will not open to follow the movement of the joint, and the seal will no longer be effective.

The joint sealant reservoir width for preformed compression seals must be designed to provide a smaller working range than for liquid sealants. Standard specifications for preformed compression seals are found in ASTM D2628.

Compression seals require extra joint preparation. Joints must be uniform in width and have perpendicular sidewalls. Spalls that extend below the top of the compression seal <u>must be repaired</u>. Such spalls will cause the seal to work its way out of the joint, resulting in complete sealant loss. Compression seals have provided adequate performance on some resealing jobs (10). Intrusion of incompressibles between the seal and the joint face has been observed in some instances. The life of compression seals in new pavements is over 10 years (73). The U.S. Air Force utilizes these seals on resealing jobs extensively, but extra spall repair at joints is often needed (106).

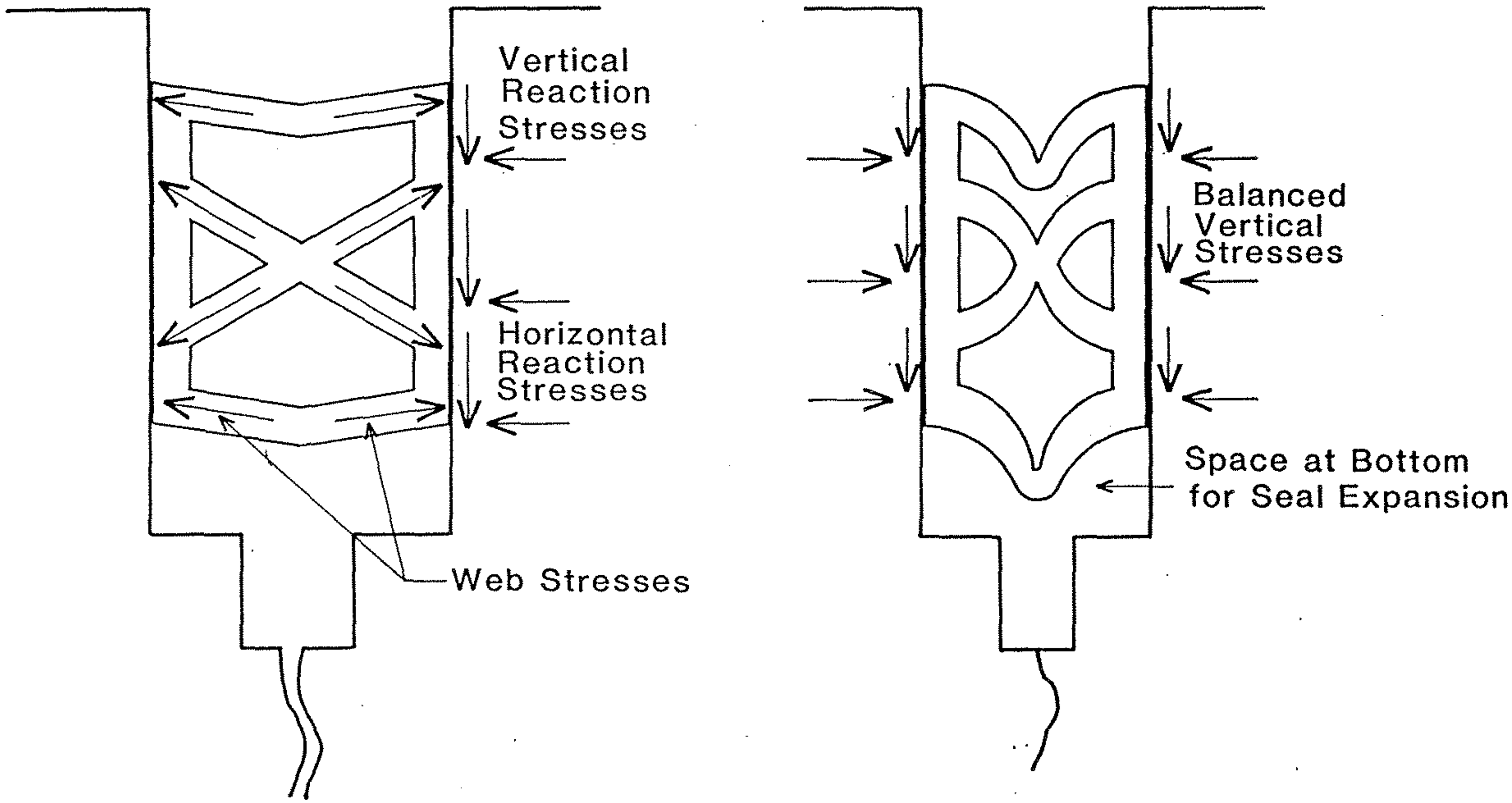


Figure 100. Preformed Compression Seal.

Lane/Shoulder Joint Sealant Considerations

Lane/shoulder joints are subject to very large vertical movements caused by differential settlements and heaves. They may also be subject to large horizontal movements when the shoulder is not adequately tied to the mainline pavement. This is especially true for PCC pavements constructed with asphalt concrete shoulders. Most sealants perform poorly in such cases, although "improved" rubber asphalt sealants, silicone sealants, and crumb rubber asphalts have been used in these joints. Large movements must be addressed in designing the joint reservoir size. The reservoir should be at least 1 in [25.4 mm] wide. NCHRP Project 14-3 developed recommendations and proposed specifications for sealing the longitudinal lane/shoulder joint (7). Another research project is currently underway by FHWA/ERES to evaluate new sealants and methods of approaching this problem.

Crack Sealing Considerations

Many cracks do not experience large movements, and sealants placed in these cracks can potentially outperform joint seals on the same project. In pavements where the cracks are the functional joints, they must withstand similar movements. The same sealants used for sealing joints should be used to seal these concrete pavement cracks. Installation and design procedures are also similar.

Unlike joints, cracks are irregular in dimension and direction, and are <u>more</u> difficult to seal. Self-levelling sealants are generally easiest to install, although tooled silicone and polymer sealants have been installed successfully. Preformed compression seals cannot be used effectively in cracks.

Other Considerations

<u>The best available joint sealant materials should be selected</u> for rehabilitation and new construction work. Joint preparation and traffic control are generally the most expensive activities in a resealing project. A slightly more expensive sealant material may extend the sealant life considerably, resulting in a lower average annual cost, thus justifying initial higher sealant cost. Even when the best sealants are used, the desired performance will be obtained only if the joint is properly prepared for sealing.

Factors Affecting Sealant Design and Performance

Factors which influence sealant performance include the movement of the joint or crack, the sealant reservoir shape, the bonding between the sealant and sidewall, the properties of the sealant and joint sealant reservoir preparation (7, 9, 20, 62, 70, 73, 93, 129, 137). All of these factors must be considered in the design and construction of a joint resealing project.

Movement

The movements at a joint or crack which influence the sealant performance include horizontal opening and closing, vertical differential deflection as a wheel load passes over the joint or crack, and the vertical differential faulting that typically accompanies pumping.

Horizontal movement is caused primarily by daily and seasonal variations in temperature. The amount of opening or closing of any discontinuity (i.e., joint or crack) depends on changes in temperature, spacing between working joints or cracks, friction between the slab and base, the condition of the joint load transfer devices, and other minor factors. For design purposes, the mean transverse joint opening (change in joint width) over a time interval can be estimated using the following expression (26).

$$DL = C\ L[a\ DT + e]$$

where:

DL = the mean change in joint width, inches;

C = an adjustment factor to account for subbase/slab friction restraint (use 0.65 for stabilized subbase, 0.80 for granular base);

L = the joint spacing, inches);

a = the thermal coefficient of contraction of expansion for Portland cement concrete ($5\text{-}6 \times 10^{-6}/^{o}F$ [$9\text{-}10.8 \times 10^{-6}/^{o}C$];

DT = the change in temperature, degrees F; and

e = the drying shrinkage coefficient of the PCC slab ($0.5\text{-}2.5 \times 10^{-4}$ strain), which can be neglected in resealing work.

The result is the mean joint opening. For a given project there are many joints and each open a different amount for any given drop in temperature.

Whereas the above expression will give an estimate of the amount of joint movements, it is recommended that actual joint movements be obtained for previously constructed projects similar in design as the one under consideration.

Differential vertical deflections occur when traffic crosses a joint. Permanent vertical movements occur when the slabs fault or when slabs or shoulders heave or sag. Vertical movement of any kind is very detrimental to joint sealant performance and large permanent settlements may cause a sealant to fail in shear.

In spite of their importance, vertical movements are not presently used as a design criterion. They should be minimized as much as possible through proper restoration (e.g., load transfer restoration, cement grout subsealing, etc.) prior to joint sealing.

Sealant Reservoir Shape

The joint sealant reservoir width must be designed to accommodate the allowable sealant strain and the depth must be sufficient to develop bond between the joint sidewall and liquid sealant or to accommodate the preformed sealant.

During periods of extreme cold, field-molded joint sealants are subjected to large tensile strains. The resulting strains may cause a liquid sealant to pull away from the sidewall or may even split the sealant itself. The shape and width of the sealant reservoirs are designed to minimize the sealant strain. (see Figure 101). Tons (129) showed that a reservoir with a depth-to-width ratio of 1:1 would minimize stresses within the sealant and along the sealant/sidewall interface. These stresses are further reduced when the upper and lower surfaces of the sealant are free to deform (i.e., three-sided adhesion is prevented through the use of a bondbreaker or backer rod at the bottom of the joint reservoir). Field and lab experience indicates that liquid sealants perform best with a shape factor of about 1:1 while silicone sealants can perform adequately with a shape factor of about 1/2:1.

Liquid sealants should not be extended by more than 20% of their original width, while silicone sealants are capable of withstanding at least 50% strain. Preformed compression seals must always be compressed to at least 80% of their uncompressed width to maintain good contact with the joint sidewalls, but must not be compressed to more than 50% of their uncompressed width or they may become permanently deformed.

Construction and longitudinal joint reservoirs should have a depth of approximately 0.75 in [19 mm] and a width of 0.25 in [6 mm], although shape factors are not usually critical since these joints are not typically subject to appreciable movement.

Longitudinal lane/shoulder joints require special attention, especially when the pavement and shoulder are dissimilar materials. Large differential longitudinal movements and permanent deformations such as settlement or heaving of the shoulder make it difficult to seal this joint effectively. Rubberized asphalt seals in joint reservoirs at least 1 in [25 mm] wide and 1 in [25 mm] deep have been used in sealing the longitudinal joint between PCC pavements and asphalt shoulders (7). When tied PCC shoulders are present, sealing this joint is similar to sealing longitudinal joints between traffic lanes.

Sealant Reservoir Preparation

No matter how good the sealant and design, the sealant will not perform as designed unless the reservoir is properly prepared. The main concern is cleanliness of the joint face. Any contamination of the joint face with old sealant, dust, or sawing residue will prevent the joint sealant from developing the needed bond with the joint face. Construction procedures and inspection must be directed at providing a clean joint face.

Construction Considerations

Joint Resealing

Joint resealing consists of the following steps:

1. Removing the old sealant.

2. Refacing the joint sidewalls to obtain an appropriate shape factor.

3. Cleaning the joint reservoir by sand blasting and air blowing.

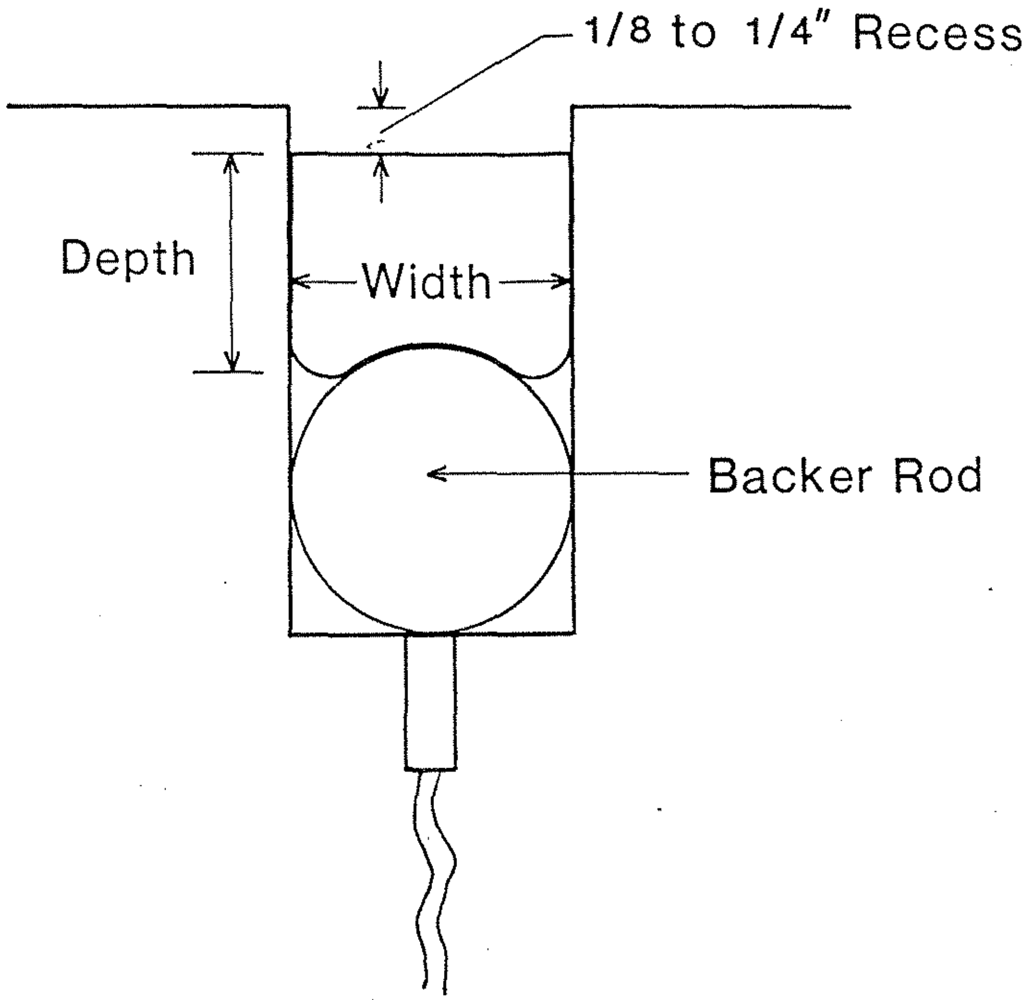


Figure 101. Joint Sealant Reservoir Dimensions [1 in = 25.4 mm].

4. Inserting the backer rod.

5. Installing the sealant.

There are several methods of removing old sealants. Liquid sealants are often cut or scraped loose using tractor-mounted vertical cutting tools or plows. Special care must be taken not to damage the joint by using a "vee" shaped plow. A rectangular-shaped plow is acceptable. High-pressure water can also be used. Preformed seals should be cut to facilitate removal. The old sealant must be removed to a depth that will accommodate the new sealant plus any backer or bondbreaker material that will be used. After scraping or cutting, all loose materials should be removed using air or water blasting.

A well-defined, uniform reservoir is essential for the optimum performance of all types of joint seals. For this reason, it is often necessary to perform partial-depth repair of spalls in the vicinity of the joint prior to placement of the new seal. This is always necessary when preformed compression seals are used. Discussion of partial-depth repair procedures is presented in another section of this report.

It is usually necessary to reface the joint to provide a well-defined reservoir of proper width and depth. Irregularities could result in poor sealant performance due to failure of the seal to remain in place. If these operations do not remove all of the old material to the desired depth and provide clean exposed concrete on the joint faces, further refacing will be required using diamond or abrasive wheel saws. These saws can also be used to widen the top portion of a joint reservoir to meet established shape and width requirements.

After the sawing operation, the joint surfaces should be cleaned by sandblasting. Sawing and refacing normally creates a slurry or dust which will adhere to the face of the joint. Only sandblasting will remove it.

After the joint has been properly prepared, a bondbreaker or backer rod material should be placed. Closed cell foams, cotton rope and nonreactive adhesive-backed tape are commonly used materials.

Methods of sealant installation vary with the type of sealant installed. Hot-poured sealants are often melted in indirect-heating double boilers. It is important that the kettle have a mechanically operated agitator (to ensure uniform heating and melting) and a positive thermostatic temperature control. These sealants are most effectively installed by injection through nozzles that have been shaped to penetrate the joint and fill the reservoir from the bottom. Hand-held pouring pots are often used on small jobs but do not provide the quality desired.

At the time of placement, the reservoir must be clean, dry and free of dust. Moisture can form steam bubbles in hot sealant and create voids when the sealant cools. The pavement temperature should be greater than 50°F [10°C] at the time of application. The top of the new seal should be 0.25 in [6 mm] below the pavement surface to prevent traffic from damaging the sealant during the warm seasons.

Cold-poured sealants are generally two-component sealants fed from separate sources and mixed just before the sealant passes through the applicator nozzle. These materials must be applied using equipment specified by the manufacturer. Placement procedures are similar to those given for the hot-poured sealant, including the 0.25 in [6 mm] placement below the pavement surface. Installation of silicone sealants requires special application equipment which forces the sealant into the joint, presses it against the joint faces and tools the surface.

Installation of preformed compression seals requires the application of a lubricant/adhesive to the joint sidewalls immediately prior to insertion of the compression seal. This material eases the insertion of the seal and cures to form a very weak adhesive which helps hold the seal in place. Joint faces should be surface-dry and air and pavement temperatures should be above 40°F [4°C]. Compression seals must be inserted carefully to avoid twisting or stretching the seal. If the seal stretches more than 5 percent, the seal will break and come out of the joint. Most manufacturers of compression seals provide installation machines or special tools to minimize this problem.

Preparation of pressure relief joints for resealing is similar to the procedure given above. Since these joints are filled with a nonextruding compressible filler material, it is necessary only to clean the old sealant down to this filler. The faces of the joint should be cleaned. Since the old filler may have absorbed some of the old sealant, it is recommended that a bondbreaker strip be used between the new sealant and the filler, especially if there is any question of compatibility between the old sealant and the new.

Inspection of the sealant project is the only method to ensure compliance with the strict standards required for sealant installation. The materials must be tested for specification compliance prior to their installation in the pavement. This testing should be done by a state or private testing laboratory, and not taken from the manufacturer's certificate of acceptance.

Crack Sealing

The variability of cracks often necessitates different repair procedures for different crack conditions. In general, the following procedures are recommended:

1. Low-severity crack, hairline to 0.125 in [3 mm] wide with no spalling: do not widen or seal.

2. Hairline to 0.125 in [3 mm] wide cracks with no spalling but some faulting: rout or saw before sealing.

3. Cracks 0.125 in to 0.375 in [3 to 10 mm] wide with no spalling: rout or saw before sealing.

4. Cracks 0.125 in to 0.375 in [3 to 10 mm] wide with minor spalling: rout or saw before sealing.

5. Cracks 0.375 in to 0.75 in [10 to 19 mm] wide with major spalling: partial-depth repair of spalls, maintaining the crack through the repair; seal the crack.

6. Cracks greater than 0.75 in [19 mm] wide with no spalling: rout and seal (consider using a backer rod if the crack is too deep).

7. Cracks greater than 0.75 in [19 mm] wide with major spalling: full-depth repair of pavement including construction of a joint.

Low-modulus silicone sealants have shown a tendency to pull away from irregular surfaces, producing failure along the entire joint or crack. If these sealants are to be used where spalls are present, the spalls must be repaired first for good performance.

CHAPTER XII

EVALUATION OF IN-SERVICE JOINT RESEALING PROJECTS

AZ017206 - This project was an experimental 1500-ft [457-m] section in the outer lane of 6-lane Interstate 17. The pavement was an undowelled 9-in [229-mm] JPCP with 15-ft [4.7-m] joint spacing which utilized different joint sealants over its length. The experimental project was constructed in 1982. Three years prior to the joint resealing, diamond grinding was performed on the pavement to improve the riding characteristics. The types of sealants used included three asphalt rubber sealants (ARCO, Sahuaro and MC250 plus rubber), a silicone sealant (Dow Corning 888 Silicone Rubber Highway Sealant) and a polymer-type, hot-poured elastomeric sealant (Superseal 444) which were installed in 25, 22, 1, 25 and 27 joints, respectively.

The project is located in the Phoenix area and is subject to high traffic levels. The resealed joints have been exposed to almost 10 million 18-kip [80-kN] equivalent single axle loads (ESAL). Current traffic levels are applying roughly 1.5 million ESALs annually to the project in the outer lane (57000 vehicles per day, 10.4% heavy commercial trucks).

The pavement showed very little distress. The most prominent distress found was transverse joint and corner spalling. There was no apparent pattern between spalling occurrence or severity and type of joint sealant. Other distresses located in the project were scaling and map cracking. The roughness of the pavement was measured using a Mays Ride Meter, obtaining a value of 144 in 1984. Surface friction was measured to be 51.

New sealant reservoirs 2.25 in [57 mm] deep and 0.50 in [13 mm] wide were formed during the sealing operation. The sidewalls were cleaned by sandblasting and a backer rod was installed to act as a bondbreaker under the sealants.

The Dow Corning silicone sealant performed the best with no deterioration evident. The MC250 plus rubber sealed joint is also intact; however, it is not practical to evaluate this sealant based on its performance in just one joint. The polymer-type sealant (Superseal 444) also performed well. The asphalt rubber sealants seemed to be incompatible with the climate of the Southwest due to the adhesion or debonding mode of failure of the ARCO and Sahuaro sealants. These failures may, in part, be due to poorly designed joint reservoir shape factors.

IA030148 - The original pavement built in 1964 was constructed as a 10-in [254-mm] JPCP at a joint spacing of 20 ft [6.1 m]. Pressure relief joints 4 in [102 mm] wide were installed at 1000-ft [305-m] intervals in 1980 and joint resealing was performed in 1984. A hot-poured sealant, "Sofseal," replaced the original asphalt sealant.

The project (a 2-lane highway) currently supports about 0.16 million ESALs annually (2385 vehicles per day, 14.3% heavy commercial trucks). Since the resealing work was performed in 1984, 0.33 million ESALs have been accumulated on the project.

The distresses displayed on this project were transverse joint and corner spalling, joint faulting, corner breaks and "D" cracking. Some medium-severity transverse cracks were noted at locations where transverse joints had not been sawed. Roughness was measured in September, 1983 by an IJK Road Meter, yielding an average Present Serviceability Index of Roughness (PSIR) of 3.23 (fair). Surface friction values of 34 (marginal) and 32 (marginal) were obtained in the eastbound and westbound lanes, respectively, in July, 1984, using a locked-wheel trailer with an ASTM E274 standard tire. Serviceablity was evaluated in September, 1983, with a PSI of 3.3 obtained for the eastbound lane and 3.2 for the westbound lane.

The joint resealing operation produced a new joint sealant reservoir 0.5 in [12.7 mm] wide and 1.0 in [25 mm] deep. Before placing the new sealant, the sidewalls were refaced with a single diamond saw blade and then sandblasted clean. A backer rod was installed to provide a bondbreaker under the sealant. The project was successful in that all of the resealed joints are well sealed, with incompressibles present in only a few joints. Spalling at the joints is generally not a problem.

NE080382 - The pavement consists of a 9-in [229-mm] JRCP with a joint spacing of 46.5 ft [14.2 mm], built in 1962. The rehabilitation work evaluated was performed in 1982 and included full-depth repairs, partial-depth repairs, 4.5-in [114-mm] wide pressure relief joints at 1-mile [1609 m] intervals, and joint resealing.

The old sealant was removed with a diamond blade saw and the sidewalls of the reservoir were cleaned by sandblasting. No information was available on the joint sealant reservoir dimensions. A hot-poured sealant conforming to ASTM specification D3405-78 (tradename Seal Tight, manufactured by W. R. Meadows) was used on this project. The sealant was placed to a depth of 0.12 in [3.0 mm] below the pavement surface. A backer rod was used as a bondbreaker. The bond of the sealant was lab-tested at 0°F [-18°C], in which the sealant was required to pass 100% extension over a period of 3 cycles. The full-depth repairs were installed to address joint spalling caused by expansion of reactive aggregates. The pressure relief joints also addressed the expansive pressure generated by the reactive aggregate expansion.

Current traffic consists of 15000 vehicles per day in each direction, including 24.0% trucks. The pavement has accumulated 7.92 million ESALs since the time of rehabilitation. Current annual traffic is equivalent to 2 million ESALs in one direction.

The pavement exhibited transverse cracking at approximately 15- to 20-ft [4-6 m] intervals. Reactive aggregates accounted for much of the observed joint spalling. A roughness value of 1090 (very rough) was obtained in June, 1984, using a Mays Ride Meter. Surface friction was measured in June, 1983 by a locked-wheel trailer with ASTM E274 standard tire. An average skid number of 33 (marginal) was obtained.

Joint sealant was present in 80% of the contraction joints; however, when absent, incompressibles were filling the joint. Joint deterioration was evident as medium-severity transverse joint spalling and corner spalling. The joint sealing operation is only moderately successful in that some of the joint sealant is missing after only 3 years of exposure and the spalling at these joints, while probably due to the reactive aggregate present in the pavement, is aggravated by the presence of incompressibles in the joints. The resealed joints are approaching the end of their life, and a sealant with a longer life should be considered.

IL080105 - The original pavement was constructed in 1960 as a 10-in [254-mm] JRCP with a joint spacing of 100 ft [30.5 m]. The 4-lane Interstate highway was rehabilitated from 1983 through 1985. The experimental techniques performed on this project included full-depth repairs, pressure relief joints, cement grout undersealing, diamond grinding, longitudinal underdrains, load transfer restoration and joint resealing. The experimental joint resealing project was located in the eastern end of the eastbound lanes. A hot-poured sealant (9001, manufactured by Allied Joint Seal and conforming to ASTM D3405-78) was used to seal the joints.

The traffic level at the project location consists of 7000 vehicles per day, including 34.0% heavy commercial trucks. This translates into a current annual application of 1.3 million ESALs in each direction (1.1 million in the outer lane) and a cumulative 3.0 million ESALs on the resealed joints (2.6 million in the outer lane).

The pavement shows signs of transverse and corner joint spalls, joint faulting, scaling and deteriorated sealant conditions. The outer lane has many more deteriorated cracks than the inner lane. The original joints, which were resealed, were all in good condition. The joints were free of incompressibles and did not exhibit significant joint spalling. Roughness was measured at 64 (very smooth) in 1985 and 82 (smooth) in 1986 using a BPR Roughometer. The surface friction was evaluated at 40 (good) using a locked-wheel trailer with ASTM E 274 standard tire. Data for calculation of Present Serviceability Index (PSI) values was not available.

The joint resealing performed in 1984 was very successful. All of the joints were well sealed, free of incompressibles and performing well. The new joint sealant reservoir was 0.38 in [10 mm] wide and 0.75 in [19 mm] deep. No bondbreaker was used. It is too early to thoroughly assess the performance of the new sealant, but it was noted that the resealed joints have less spalling than the unsealed ones.

LA010151 - This Interstate highway was constructed in 1971 as a 10-in [254-mm] JRCP with a joint spacing of 58.5 ft [17.8 m]. This project was rehabilitated in 1984 as part of Federal Highway Administration Demonstration Project No. 69, "Portland Cement Concrete Pavement Restoration" (24). The rehabilitation techniques applied to the pavement were full-depth repairs, cement grout undersealing, load transfer restoration, diamond grinding, joint resealing, crack repair and longitudinal underdrains.

The old joint sealant was removed and the joint sidewalls were refaced with a two-bladed diamond saw. The new joint sealant reservoir was sawed to a depth of 1.50 in [38 mm] and width of 0.50 in [13 mm]. The joint sidewalls were cleaned by sandblasting to remove sawing residue and other contaminants prior to placement of the sealant material. A backer rod was placed under the sealant as a bondbreaker. Four silicone sealants were used on this rehabilitation project. The silicones were manufactured by General Electric, CSC Silicones Ltd., SWS Silicones Corporation, and DOW Corning Corporation. The longitudinal joint and the lane/shoulder joint were sealed with two types of rubberized asphalt joint sealant ("Hi-Spec" and "Sof-Seal") manufactured by W. R. Meadows, Inc. Information on the joint sealant reservoir dimensions and preparation were unavailable.

The traffic level of this project consists of 16000 vehicles per day, including 21.0% heavy commercial trucks, which is equivalent to a current annual loading of 1.4 million ESALs in the driving lane and 0.4 million in the passing lane. The accumulated traffic loading on the rehabilitation techniques is 2.79 million ESALs in the outer lane and 0.93 million ESALs in the inner lane.

The pavement exhibited transverse joint and corner spalling, joint faulting, pumping and transverse and longitudinal cracking. Pumping was more predominant near slabs which exhibited medium-severity transverse and longitudinal cracking. All of the remaining original contraction joints exhibited either medium-severity transverse joint spalling or corner spalling and faulting. Surface friction was measured three months before and one month after rehabilitation. A skid number of 40 was obtained at both times using a locked wheel trailer with ASTM E274 standard tire. Roughness measurements before and after the rehabilitation were unavailable. However, the PSI after the rehabilitation was 4.4, compared to 4.1 before the rehabilitation (outer lane only).

All the transverse joints were cleaned and resealed in October, 1984. The sealant placed in 1984 was present in all of the joints. No incompressibles were present. Cohesive failure was present in 11 of the 64 joints (17%) that were surveyed and adhesive failure of the sealant was observed in only one of the 58 joints (2%) surveyed. The cohesive failures (tensile failure within the silicone joint sealant) may be an indication that the sealant thickness was too small. The following table shows the number of cohesive failures by sealant type:

Sealant Type	Number of Cohesive Failures
General Electric	8
CSL Silicones Ltd.	1
SWS Silicones Corporation	1
Dow Corning Corporation	1

IL280-74 - The original pavement was a 10-in [254 mm] JRCP with 100-ft [30.5 m] contraction joints, constructed in 1961 and opened to traffic in 1962. Rehabilitation techniques applied include full- and partial-depth repairs, cement grout undersealing, diamond grinding, longitudinal underdrains, and joint resealing.

A joint resealing program was conducted on the project in 1984. All of the regular contraction joints were resealed in conjunction with the full-depth repair placement. The joints were routed to a depth of 1 in [25 mm] and to a width of 0.625 in [16 mm]. The joints were then sandblasted and airblown clean prior to sealing with a hot-poured sealant conforming to ASTM D 3405. No bondbreaker material was used. Cracks were also resealed with the same hot-poured sealant (ASTM D 3405).

Traffic levels on the project vary from I-280 to I-74. On I-280, 8000 vehicles per day, including 20% trucks, are producing 1.2 million ESALs annually and have accumulated almost 2 million since the joint resealing was performed. On I-74, 4000 vehicles per day, including 25% trucks, are producing 0.7 million ESALs annually and have accumulated 2 million since the joint resealing was performed.

The pavement exhibited transverse joint and corner spalling, joint faulting, localized scaling and deteriorated sealant conditions. Approximately 47% of the outer lane regular contraction joints and 40% of the inner lane regular contraction joints exhibited medium-severity transverse joint or corner spalling. In almost every case, these spalls were found along joints that had damaged sealant. Faulting of the contraction joints was minimal, probably due to the fact that diamond grinding was performed only one year prior to the survey. Roughness was measured at 40 (very smooth) in 1985 using a BPR Roughometer and surface friction was measured at 42 (good) the same year using a locked-wheel trailer with an ASTM E274 standard tire.

The joint resealing program performed in 1984 is not considered successful because the sealant failed to adhere to the existing concrete. The joint sealant is partially or completely absent from most (89%) of the joints. As a result, incompressibles and water can freely enter the joints and cause pumping and spalling. Transverse joint and corner spalling was found at nearly half of the surveyed regular contraction joints. The probable cause for the sealant failure was the joint reservoir not having a properly designed shape factor to allow for the amount of contraction occurring in the 100-ft [30.5 m] slabs.

OH077053 - The original pavement was a 9-in [229 mm] JRCP with 60.0-ft [18.3 m] contraction joints, constructed and opened to traffic in 1967. The project was an experimental section which provided a field study of the techniques developed in NCHRP Project 1-21, specifically full- and partial-depth repairs, undersealing, load transfer restoration, diamond grinding, and joint resealing (37). All rehabilitation techniques were performed in the outer lane only.

The outer lane joints were resealed in 1982. The old sealant was removed and the sidewalls were refaced by a two-bladed diamond saw. New joint sealant reservoirs were produced with a depth of 1 in [25.4 mm] and a width of 0.5 in [12.7 mm]. Three different types of joint sealant were used in this project: a hot-poured sealant made in accordance to ASTM specification D3405-78, preformed compression seals (ASTM specification D2628-81), and a low-modulus silicone sealant. The hot-applied rubberized asphalt was manufactured by Posh Chemical Company, the preformed joint seals by D.S. Brown Company, and the silicone was manufactured by General Electric.

Backer rods were placed under the sealant as a bond breaking material except where the neoprene preformed seals were used. The sealants were placed 0.25 in [6.4 mm] below the pavement's surface. The hot-poured sealant was used in most of the joints and in all of the cracks that were sealed. Only four joints were sealed with the silicone and two were sealed with the preformed compressive sealants.

Traffic levels at the project site consist of 4750 vehicles per day, including 25% heavy commercial trucks, which translates into a current annual application of 0.6 million ESALs in the driving lane and 0.1 million in the passing lane. The accumulated traffic loading on the resealed joints totals 2.1 million ESALs in the driving lane and 0.5 million ESALs in the passing lane.

The project has experienced some distress such as transverse joint and corner spalling, joint faulting, longitudinal and transverse cracking and deteriorated sealant condition. Where spalling was observed, it was of low severity. Longitudinal joint spalling was not observed in the repairs. Transverse cracks were observed extending across both lanes of the pavement at approximately 15- to 20-ft [4-6 m] intervals. The sealant was absent in some of the original inner lane contraction joints and varying amounts of incompressibles were present in both lanes. The transverse joint spalling and corner spalling were slightly worse in the inner, unrepaired lane. Roughness was measured at 68 (smooth) in July, 1984 using a G.M. Profilometer. Surface friction was measured at 45 (good) in November, 1984 by a locked wheel trailer with a ASTM E274 standard tire. A Pavement Serviceability Rating of 3.1 was obtained in June, 1984.

The joint resealing project was successful, as most of the joints are still sealed, and incompressibles are not present. Spalling is not a problem throughout the project. In terms of the sealant itself, several localized distresses are present, such as material loss and bond or sealant failure. This is true only for the hot-poured sealants, as the silicone and neoprene sealant are still in good condition.

NE080279 - This pavement was constructed in 1963 as a 9-in [229-mm] JRCP with a joint spacing of 46.5 ft [14.2 m]. The rehabilitation techniques applied to this project in 1982 included full-depth repairs, epoxy cement partial-depth repairs, 4.5-in [114-mm] wide pressure relief joints, and joint resealing. These techniques addressed joint and crack spalling and pressure buildup caused by expansion of reactive aggregate.

In June, 1982, the joints were resealed. The old sealant was removed with a diamond blade saw and the sidewalls of the reservoir were cleaned by sandblasting. No information on the new joint reservoir dimensions was available. A hot-poured sealant conforming to ASTM specification D3405-78 (manufactured by Crafco, Inc.) was used on this project. The sealant was placed at a depth of 0.12 in [3.0 mm] below the pavement surface. No bondbreaker was used. The bond of the sealant was lab-tested at $0^{o}F$ [$-18^{o}C$], during which the sealant was required to pass 100% extension over a period of 3 cycles.

The traffic at this site is composed of 9600 vehicles per day, including 31% heavy commercial trucks, which corresponds to a current annual application of 1.3 million ESALs in the outer lane and 0.3 million in the inner lane. The accumulated ESALs on the resealed joints are 4.62 million in the outer lane and 1.54 million in the inner lane.

Distresses observed on the pavement included transverse joint and corner spalling, joint faulting, transverse cracking, deteriorated sealant conditions, localized scaling and reactive aggregate distress. Transverse cracking seemed to occur at intervals of 15 to 20 ft [3 to 6 m]. Some panels are in need of immediate repair due to high levels of deterioration. This project does not exhibit the amount or severity of reactive aggregate distress observed on other Nebraska projects. The contraction joint sealant was present in most of the original contraction joints, keeping incompressibles from infiltrating. As a result, transverse joint spalling and corner spalling were of low severity throughout the surveyed sample units and none of the surveyed original contraction joints had been repaired.

Roughness was measured in June, 1984 by a Mays Ride Meter. An average Roughness Index of 1384 (very rough) was obtained for both lanes. Surface friction was measured in June, 1983 by a locked-wheel trailer with ASTM E274 standard tire. An average skid number of 39 (marginal-good) was obtained for both lanes.

The transverse joint sealant was still intact in most of the transverse joints and incompressibles were not observed in these joints. The only joints exhibiting significant sealant deterioration were those that had not been resealed in 1982. The sealant had extruded from these joints. Spalling is generally not a problem. From these observations, the resealing operation can be considered a success.

NE080404 - The pavement is a 10-in [254-mm] thick JPCP with a joint spacing of 16.3 ft [5 m], constructed and opened to traffic in 1960. Rehabilitation of the pavement in 1984 consisted of full-depth repairs, epoxy cement partial-depth repairs and joint resealing. Rehabilitation was performed to correct joint spalling and slab cracking.

The old sealant was removed with a diamond blade saw and the sidewalls of the reservoir were cleaned by sandblasting. No information is available on the joint reservoir dimensions. A hot-poured sealant conforming to ASTM specification D3405-78 (tradename Crafco RS-231, manufactured by Crafco Inc.) was used on this project. The sealant was recessed to a depth of 0.12 in [3.0 mm] below the pavement surface. A backer rod was used as a bondbreaker under the sealant. The bond of the sealant was lab-tested at 0°F [-18°C], during which the sealant was required to pass 100% extension over a period of 3 cycles.

Current traffic consists of 14100 vehicles per day, including 21% heavy commercial trucks, equivalent to 1.2 million ESALs annually in the driving lane and 0.4 million in the passing lane. Since the rehabilitation work was performed in 1984, 2.47 million ESALs have been accumulated in the outer lane and 0.82 million in the inner lane.

Distresses observed included transverse and longitudinal cracking, transverse joint and corner spalling, joint faulting, localized scaling and deteriorated joint sealant condition. Transverse slab cracking was minimal, but approximately 250 linear ft [76 m] of low- and medium-severity longitudinal cracking was observed in the 3000 ft [914 m] of surveyed pavement. The cracking generally originated near and followed the pavement centerline, indicating poor longitudinal joint construction techniques. The contraction joint sealant was present in all of the original contraction joints. However, the sealant had extruded in several of these joints and incompressibles had infiltrated. Medium-severity transverse joint and corner spalling were observed at some joints, but were not a major problem.

Roughness was measured in June, 1984 using a Mays Ride Meter. An average roughness Index of 1151 (very rough) was obtained for the outer lane, and 1617 (very rough) for the inner lane. Surface friction was measured in June, 1983 by a locked-wheel trailer with an ASTM E274 standard tire. An average skid number of 32 (marginal) was obtained.

The joint resealing project has been successful in preventing water and incompressibles from entering the transverse joints. Insufficient recessing of the sealant has resulted in some extrusion of the sealant however. Where the sealant was extruded, incompressibles have infiltrated the joint. The longevity of the sealant is still under evaluation.

VA044000 - The pavement was built in 1967 as a 9-in [229-mm] JRCP with a transverse joint spacing of 61.5 ft [18.7 m]. Rehabilitation performed has included full-depth repairs in 1984, partial-depth repairs and pressure relief joints in 1984 and 1976, and joint resealing in 1984, 1976 and 1973. The 1976 resealing program replaced the 1973 program, and was performed only on part of the project. Both the 1973 and 1976 resealing programs used preformed joint seals. The joints were resawed to a minimum of 0.625 in [16 mm] wide x 2.25 in [57 mm] deep and then cleaned thoroughly. The preformed joint seals were then placed. At this time, the longitudinal joint was also resealed with a hot-poured sealant.

The 1984 joint resealing program was aimed only at selected joints which were demonstrating joint deterioration. The old sealant was removed and the joint was thoroughly cleaned. New hot-poured sealant was then placed in the joint. The joint sealant reservoir dimensions are not known. The longitudinal joint was also resealed at that time.

Current traffic consists of 29600 vehicles per day, including 4.7% trucks, equivalent to 1.05 million ESALs on the 1984 joints, 4.03 million on the 1976 joints and 4.88 million on the 1973 joints. Currently, an annual loading of 0.4 million ESALs is applied to the outer lane and 0.2 million in the inner lanes.

Distresses observed included transverse joint and corner spalling, joint faulting and localized scaling of the pavement. The amount and severity of slab cracking varied substantially from one sample unit to the other, with as few as 75 low-severity transverse cracks per mile [47 cracks/km] in one sample unit to as many as 352 total transverse cracks per mile [220 cracks/km] in another sample unit. The joint sealant was present in all of the original contraction joints sealed with preformed compression seals (first and second sample units). However, incompressibles were frequently found in those joints, even though the compression seals were still intact. The transverse and longitudinal joints sealed with the hot-poured sealant were still sealed and performing well. Most of the remaining original contraction joints exhibited low-severity transverse joint spalling, with a few displaying medium-severity spalling.

Roughness was measured in 1980 with a Mays Ride Meter. A Roughness Index of 109.8 (good) was obtained in the eastbound truck lane, and a Roughness Index of 116.4 (good) was obtained in the westbound truck lane. Surface friction values were not available.

The joint resealing projects are considered fairly successful. The sealant is still intact, and spalling is not a problem throughout the project. The preformed seals were placed in 1976 and have experienced some localized failures where the joint has opened sufficiently to cause the sealant to lose contact with the reservoir walls, drop down and allow incompressibles and water to enter. This type of failure is attributed to the relief joints, although a low-modulus adhesive sealant might have performed without failure under these conditions. The hot-poured sealant (placed in the third sample unit during 1984) is performing well and there are no incompressibles present. The sealant is extruded in some locations, indicating improper recessing of the sealant within the reservoir.

VA064202 - The pavement was constructed in 1966 as a 9-in [229-mm] JRCP with a transverse joint spacing of 61.5 ft [18.7 m]. Rehabilitation performed on the pavement includes full- and partial-depth repairs in 1984 and 1976, pressure relief joints in 1982 and joint resealing in 1984 and 1976. Both of the resealing programs used preformed joint seals. The joints were resawed to a minimum of 0.625 in [16 mm] wide x 2.25 in [57 mm] deep and then cleaned thoroughly before the preformed joint seals were placed. At that time, the longitudinal joint was also resealed with a hot-poured sealant.

The traffic at the project site is composed of 10000 vehicles per day, including 15% heavy commercial trucks, which has accumulated 1.08 million ESALs on the 1984 joints and 4.09 million on the 1976 resealing job. Currently, an annual loading of 0.4 million ESALs is applied to the outer lane and 0.1 million to the passing lane.

The pavement exhibits transverse joint and corner spalling, joint faulting, localized scaling and deteriorated joint sealant conditions. Mainly low-severity slab cracking was observed and only in the outer lane.

The preformed joint seals are still present in all of the regular contraction joints (both lanes in both directions). However, incompressibles were found in roughly 70% of the joints. Joint widths averaged 0.83 [21 mm] for the joints with incompressibles and 0.70 in [18 mm] for the remaining regular contraction joints. Contraction joints located near pressure relief joints and full-depth repairs were generally slightly wider than average. Most of the original contraction joints exhibited at most low-severity transverse joint spalling, although a few joints displayed medium-severity spalling. Roughness and surface friction measurements are not available for this section.

The joint resealing project performed in 1984 is considered to be performing well. The sealant is still intact and spalling is not a problem anywhere within the project. The seals appear to have slipped down in many of the joints, however, with the result that incompressibles have collected in the reservoir on top of the preformed sealant in many cases. This indicates that the joint openings have at least occasionally exceeded the extension capability of the preformed seals. The excessive opening is probably due to the presence of so many expansion joints on this project. Spalling problems could develop in the future from this situation.

The hot-poured sealant used in the new 1984 repairs is performing well, and there are no incompressibles present. Occasionally, however, the sealant was extruded from the joint.

VA064279 - This pavement was constructed in 1967 as a 9-in [229-mm] JRCP with a contraction joint spacing of 61.5 ft [18.7 m]. Rehabilitation performed in 1981 consisted of full-depth repairs, pressure relief joints and joint resealing. The project location was also diamond ground three years after the other rehabilitation techniques were applied but only in the eastbound direction. The westbound lanes were left unground for comparison. All of the regular contraction joints were resealed in conjunction with the full-depth repair placement. The joints were thoroughly cleaned by airblasting, routing, and brushing to assure that the joint was free of oil, grease, existing joint material, and other foreign material. A hot-poured sealant was applied to the joint. The longitudinal joint and the lane/shoulder joint were also sealed at this time with the same hot-poured sealant.

Current traffic consists of 30000 vehicles per day, including 10.5% trucks, equivalent to an annual loading of 0.7 million ESALs in the outer lane, 0.4 million in the center lane and 0.1 million in the inner lane. The resealed joints have accumulated 5.48 million ESALs.

Distresses observed included transverse joint and corner spalling, joint faulting and localized scaling. Nearly all of the transverse cracking observed in the unground section was located in a 100-ft [30.5-m] section of pavement. All of the regular contraction joints were still well sealed. No incompressibles were found in the joints and the joints exhibited only low-severity spalling. There was no difference in spalling between the two sample units.

Surface friction measurements were not available. However, roughness was measured in early 1985 using a Mays Ride Meter. The results were:

LANE	ROUGHNESS
Eastbound outer (ground)	72 (excellent)
Eastbound center (ground)	66 (excellent)
Eastbound inner (ground)	64 (excellent)
Westbound outer (not ground)	145 (fair)

The joint resealing operation performed in 1981 is considered successful. The joint sealant is still intact and the joint is free of incompressibles. Spalling was not a problem in the surveyed sample units.

VA081147 - The pavement was constructed in 1965 as a 9-in [229-mm] JRCP with a transverse joint spacing of 61.5 ft [18.7 m]. Rehabilitation techniques performed in 1984 included a CPR project in the first sample unit, consisting of full-depth repairs, partial-depth repairs, cement grout undersealing, diamond grinding, longitudinal underdrains, load transfer restoration, and joint resealing. In the second sample unit, rehabilitation consisted of pressure relief joint installation, cement grout undersealing, and joint resealing.

Joint resealing was performed in 1972, 1976, and 1984, and more was scheduled for 1985. The first sample unit contained joints sealed in the 1984 program, while the second sample unit contained joints sealed in the 1976 program. The 1984 resealing program used silicone joint sealant and was performed after the diamond grinding. The joints were cleaned by routing, brushing, sawing, grinding, and airblasting. A backer rod was installed prior to the placement of the silicone sealer. The longitudinal joint was also resealed with silicone. The 1976 program used preformed joint seals, placed in joints resawed to a minimum of 0.625 [16 mm] wide x 2.25 in [57 mm] deep. The longitudinal joint was resealed with a hot-poured sealant.

Current traffic consists of 10000 vehicles per day, including 26.3% heavy commercial trucks, equivalent to 0.8 million ESALs to the outer lane and 0.2 million to the inner lane annually. The 1984 rehabilitation has been subjected to approximately 1.86 million ESALs in each direction.

Distresses observed included transverse joint and corner spalling, pumping, joint faulting, localized scaling and deteriorated sealant conditions. Transverse cracking was more severe in the second sample unit which had not received full-depth repairs. The last 200 ft [60 m] of the first sample unit and the first 300 ft [91 m] of the second sample unit exhibited much less cracking than the rest of the surveyed pavement. Roughness was measured in 1985 using a Mays Ride Meter. A roughness index of 156 (fair) was obtained in the southbound (diamond-ground) truck lane. Surface friction and Present Serviceability Index (PSI) values were not available.

The joint resealing project performed in 1984 is considered successful. The silicone sealant is still intact, spalling is not a problem, and it appears that the sealant will last many more years. The preformed seals placed in 1976 located in the second sample unit have allowed incompressibles in the joints after nine years of service. The seals appear to have slipped down into the joint reservoir in several locations, indicating that reservoir/sealant size combination used on this project may have been inappropriate for the slab size and ambient temperature range. Other possibilities include poor installation techniques and reduction in sealant flexibility over time, which could also cause the sealant to fail to follow joint movement.

VA095000 - This pavement was built in 1963 as a 9-in [229-mm] JPCP with a transverse joint spacing of 20 ft [6.1 m]. Rehabilitation has included partial-depth repairs in 1984 and 1983 and full-depth repairs and joint resealing in 1984. As part of the rehabilitation project, the transverse, longitudinal and shoulder joints were routed, brushed, blown clean and resealed with a hot-poured sealant.

Current traffic consists of 10000 vehicles per day and 20% heavy commercial trucks, equivalent to 0.6 million ESALs annually in the driving lane and 0.1 million in the passing lane. The resealed joints have received 1.51 million ESALs.

Distresses observed included pumping, joint faulting, localized scaling and transverse joint and corner spalling. Only one joint was not full- or partial-depth repaired in the outer lane, while 60 joints were unrepaired in the inner lane. All the contraction joints (both lanes in both directions) were still well sealed and apparently performing well. Incompressibles were rarely found in the joints, although medium-severity spalling was occasionally identified. Roughness was measured in 1985 using a Mays Ride Meter. A roughness index of 149 (fair) was obtained for the northbound truck lane, and a roughness index of 155 (fair) were obtained for the southbound truck lane. Surface friction and Present Serviceability Index (PSI) values were not available.

The resealed joints are performing well after one year of service. The sealant is still intact and keeping incompressibles from infiltrating.

CHAPTER XIII

PANEL CONCLUSIONS AND RECOMMENDATIONS

PERTAINING TO THE DESIGN AND USE OF JOINT RESEALING TECHNIQUES

Introduction

Fourteen joint resealing projects were evaluated. Only one of these projects (Arizona) was solely a joint resealing project, while the remaining projects were resealed in conjunction with some other form of pavement rehabilitation (such as full-depth repair, partial-depth repair and/or pressure relief joint installation). A total of 20 joint sealant installations were evaluated. Although sealant installations were identified on several additional projects, these were not installed at enough joints to make an adequate evaluation.

The typical modes of sealant failure were:

1. Adhesion - debonding of sealant from the joint sidewalls.
2. Cohesion - splitting within the sealant itself.
3. Extrusion - sealant which has been partially or completely forced from the joint onto the adjacent pavement.
4. Absence - more than 25% of the joint sealant is missing.
5. Oxidation - hardening of the joint sealant.
6. "Drop down" - slippage of preformed compression seals downward from their original recess position.

The following were typical reasons for sealant failure:

1. Inadequate joint shape factor.
2. Insufficient recessing of sealant.
3. Too many expansion/pressure relief joints.
4. Inadequate cleaning of the joint walls.
5. Inadequate use and installation of backer rod.

Observations

1. Four silicone sealant projects (located in Arizona, Louisiana, Ohio and Virginia) were surveyed. Three of these projects were successful. Some had been installed as early as 1982 and had sustained as many as 10 million 18-kip [80-kN] ESALs. The Virginia project failed in the winter following the field survey and is no longer sealed. The sealant failed due to an incompatibility between the silicone sealant and the

aggregates in the pavement. The three successful silicone projects were all installed on pavements which contained no pressure relief joints (see Figure 102).

2. The twelve hot-poured (ASTM D3405) sealant projects were located in Arizona, Illinois, Iowa, Nebraska and Virginia. Nine of the 12 projects were successful. The sealants had been installed as early as 1981 and subjected to as many as 10 million ESALs. Failures were due to inadequate joint shape factor and insufficient sealant recessing. The failures have occurred on projects resealed as recently as 1984 with only 1 million ESALs applied to the resealed joints.

3. Three of the four preformed neoprene sealant projects were "unsuccessful" because inappropriate pressure relief joint installation on these projects resulted in excessive joint openings of the nearby joints (see Figure 102). This opening caused the seals to slip down in their reservoirs and allowed incompressibles to accumulate above the seals. It should be noted, however, that the seals themselves are intact and keeping incompressibles from filtering deeper into the joints below the seals. The failures have occurred on projects resealed as late as 1984 with only 1 million ESALs applied to the resealed joints. The one successful project was located in Ohio on a pavement which contained no relief joints (see Figure 102). However, these seals were only installed on two consecutive joints.

The observed performance of the three types of joint sealants is summarized in Table 15.

4. Five of the six sealant projects installed on short-jointed pavements (joint spacing less than or equal to 30 ft [9.1 m]) were successful. The one failure was attributed to incompatibility of the hot and dry Southwest climate with asphalt rubber hot-poured sealants.

5. Nine of the fourteen long-jointed pavement resealing projects were successful. Three of the five failures contained the preformed compression seals, which failed as described previously. The two remaining failures resulted from the use of an improper shape factor. The comparison of joint spacing and performance of the joint resealing projects is summarized in Table 16.

6. When pressure relief joints have been installed on a particular pavement, the use of a hot-poured sealant has provided the best performance.

7. Projects receiving a good joint resealing program throughout their life generally appear to be in better condition than those projects receiving little joint maintenance.

Recommendations

1. The tendency on past experimental joint resealing projects has been to install different sealant materials in alternating joints or in only a few consecutive joints. Future experimental joint resealing projects should allow for more statistically significant analysis by installing

SEALANT	STATUS	USE OF PRJ: NO	YES AND APPROPRIATE	YES BUT NOT APPROPRIATE
SILICONE	SUC	3	0	0
	UNSUC	0	0	1
HOT-POURED	SUC	3	2	4
	UNSUC	3	0	0
PREFORMED	SUC	1	0	0
	UNSUC	0	0	3

Figure 102. Joint Resealing Project Performance as a Function of Sealant Type and the Use and Appropriateness of Pressure Relief Joints.

Table 15. Joint Resealing Project Performance by Sealant Type [1 ft = 0.3 m].

	PROJECT ID	YEAR OF INSTALLATION	JOINT SPACING	SHAPE FACTOR	PRJ USE *	STATUS
LOW-MODULUS SILCONE SEALANTS	AZ017206	82	15.0	4.50	NO	SUC
	LA010151	84	58.5	2.00	NO	SUC
	OH077053	82	60.0	2.00	NO	SUC
	VA081147	84	61.5	3.60	Y-NOT APP	SUC
HOT-POURED SEALANTS (ASTM D3405)	AZ017206	82	15.0	4.50	NO	SUC
	AZ017206	82	15.0	4.50	NO	UNSUC
	IL280-74	84	100.0	1.60	NO	UNSUC
	IL080105	84	100.0	1.90	Y-NOT APP	SUC
	IA030156	84	20.0	2.00	Y-NOT APP	SUC
	NE080279	82	46.5	1.00	Y-APP	SUC
	NE080382	82	46.5	1.00	Y-APP	SUC
	NE080404	84	16.3	1.00	NO	SUC
	OH077053	82	60.0	2.00	NO	UNSUC
	VA044000	84	61.5	3.60	Y-NOT APP	SUC
	VA064279	81	61.5	3.60	Y-NOT APP	SUC
	VA095000	84	20.0	3.60	NO	SUC
PREFORMED COMPRESSION SEALS	OH077053	82	60.0	2.00	NO	SUC
	VA044000	76	61.5	3.60	Y-NOT APP	UNSUC ++
	VA064202	84	61.5	3.60	Y-NOT APP	UNSUC ++
	VA081147	76	61.5	3.60	Y-NOT APP	UNSUC ++

* NO = PRESSURE RELIEF JOINTS NOT USED
* Y-APP = PRESSURE RELIEF JOINTS USED AND APPROPRIATE
* Y-NOT APP= PRESSURE RELIEF JOINTS USED BUT NOT APPROPRIATE

++ = CAUSED BY PRJ CLOSURE, SEALANT STILL INTACT

JOINT SHAPE FACTOR = DEPTH / WIDTH

Table 16. Joint Resealing Project Performance by Joint Spacing [1 ft = 0.3 m].

	PROJECT ID	YEAR OF INSTALLATION	JOINT SPACING	SEALANT TYPE #	SHAPE FACTOR	PRJ USE *	STATUS
SHORT-JOINTED	AZ017206	82	15.0	SILIC	4.50	NO	SUC
	AZ017206	82	15.0	HOT-P	4.50	NO	SUC
	AZ017206	82	15.0	HOT-P	4.50	NO	UNSUC
	NE080404	84	16.3	HOT-P	1.00	NO	SUC
	IA030156	84	20.0	HOT-P	2.00	Y-NOT APP	SUC
	VA095000	84	20.0	HOT-P	3.60	NO	SUC
LONG-JOINTED	NE080279	82	46.5	HOT-P	1.00	Y-APP	SUC
	NE080382	82	46.5	HOT-P	1.00	Y-APP	SUC
	LA010151	84	58.5	SILIC	2.00	NO	SUC
	OH077053	82	60.0	SILIC	2.00	NO	SUC
	OH077053	82	60.0	HOT-P	2.00	NO	UNSUC
	OH077053	82	60.0	PREFD	2.00	NO	SUC
	VA044000	84	61.5	HOT-P	3.60	Y-NOT APP	SUC
	VA044000	76	61.5	PREFD	3.60	Y-NOT APP	UNSUC
	VA064202	84	61.5	PREFD	3.60	Y-NOT APP	UNSUC
	VA064279	81	61.5	HOT-P	3.60	Y-NOT APP	SUC
	VA081147	84	61.5	SILIC	3.60	Y-NOT APP	UNSUC
	VA081147	76	61.5	PREFD	3.60	Y-NOT APP	UNSUC
	IL280-74	84	100.0	HOT-P	1.60	NO	UNSUC
	IL080105	84	100.0	HOT-P	1.90	Y-NOT APP	SUC

* NO = PRESSURE RELIEF JOINTS NOT USED
* Y-APP = PRESSURE RELIEF JOINTS USED AND APPROPRIATE
* Y-NOT APP= PRESSURE RELIEF JOINTS USED BUT NOT APPROPRIATE

HOT-P = HOT-POURED SEALANT
PREFD = PREFORMED COMPRESSION SEALS
SILIC = LOW-MODULUS SILICONE SEALANT

JOINT SHAPE FACTOR = DEPTH / WIDTH

the various sealants over a greater length of consecutive joints (at least a mile [1.6 km]). Different sealants could be placed in the opposing traffic lanes of the same divided highway for comparison purposes. This would allow evaluation of the performance of various joint sealants while holding all other variables relatively constant.

2. The following joint preparation steps are recommended to increase the chances for success on joint resealing:

   - Resawing of the joint reservoir rather than routing.
   - Joint cleaning performed by sandblasting, concentrating on the joint sidewalls.
   - Use of a backer rod under the joint sealant.
   - Proper installation of the backer rod.

3. Joint resealing is required for working joints to keep incompressibles out of the joint reservoir.

4. Since longitudinal joints do not undergo the same degree of movement as transverse joints, a width of 0.25 in [6 mm] is sufficient when resealing the longitudinal joint.

5. A tolerance must be set up to provide greater quality control of resealing projects. This tolerance, on both saw cuts and joint shape factor, must be developed by each agency. A good example of this concept is that applied by the Kentucky Department of Highways for their silicone rubber seals used in concrete pavements:

Joint Tolerances:	Saw Cut Depth	0 in to +0.5 in [13 mm]
	Saw Cut Width	0 in to +0.0625 in [2 mm]
	Sealant Thickness	0 in to +0.125 in [3 mm]

   The construction quality is verified by randomly selecting joints at which 5 plugs are pulled each working day. The Engineer tests each plug to see which joints are deficient with regard to the above geometrics. The Contractor removes and reworks all deficient joints and repairs the sample plug holes during the next working day.

6. When existing joint sealant conditions are poor and joint spalling exists, cleaning and resealing of all joints and cracks on the project should be performed concurrently with partial-depth repair of spalls.

7. The joint shape factor should be designed considering the type of sealant, joint spacing of the pavement, thermal coefficient of expansion of the pavement and climate. The friction factor of the subbase and any restraint provided by dowels could also be very significant in the amount of joint movement. Actual joint movements should be measured on projects similar in design to the one under consideration to determine the movements that need to be accommodated.

8. A good joint resealing program is recommended throughout the life of the pavement to help provide better performance.

CHAPTER XIV

SUMMARY

The objectives of this study were to identify, define and document the criteria for using pressure relief joints and other joint rehabilitation techniques and to provide a set of guidelines for the design, installation and use of these techniques in a pavement management system.

These objectives were accomplished by conducting a thorough review of literature, identifying a total of 36 suitable in-service study projects, collecting design (original and rehabilitation), traffic, climatic and performance data for these projects, summarizing these data in the form of individual reports for each project, presenting these reports and overall summaries to a panel of experienced state DOT personnel for their consideration and soliciting their collective conclusions and recommendations.

The major findings, recommendations and conclusions of this study for each of the four rehabilitation techniques considered are presented in separate chapters following the background and evaluation chapters for their respective techniques, as follows:

Chapter IV - Pressure Relief Joint Conclusions and Recommendations

Chapter VII - Full-Depth Repair Conclusions and Recommendations

Chapter X - Partial-Depth Repair Conclusions and Recommendations

Chapter XIII - Joint Resealing Conclusions and Recommendations

New or updated Design and Construction Guidelines and Guide Specifications for each of the four techniques are contained in the Appendices. These guidelines include new pavement management-oriented flow diagrams or decision trees to assist the user in the selection of appropriate repair techniques for individual joints and to determine the need for pressure relief joints on a given project.

A simple cost analysis was performed on the rehabilitation techniques used on selected projects. Generally, it was noted that those projects with a high annual cost were also those projects whose rehabilitation was either unsuccessful or unwarranted (e.g., the installation of pressure relief joints when not necessary often led to increased deterioration of the pavement). Conversely, the projects with a low annual cost were generally successful and sufficiently addressed the needs of the pavement. The cost analysis is presented in the Appendices.

The individual project summaries compiled for each of the surveyed projects total more than 1500 pages and are not included in this report. They are available upon request from the Federal Highway Administration.

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