DEVELOP STATEWIDE RECOMMENDATIONS FOR APPLICATION OF PCC JOINT REFLECTIVE CRACKING REHABILITATION STRATEGIES

A Thesis

by

RAHUL JAIN

Submitted to the Office of Graduate Studies of
Texas A&M University
in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

August 2004

Major Subject: Civil Engineering

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Approved as to style and content by:

Dan G. Zollinger (Chair of Committee)	Dallas N. Little (Member)
Larry J. Ringer (Member)	Paul N. Roschke (Head of Department)

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ABSTRACT

Develop Statewide Recommendations for Application of PCC
Joint Reflective Cracking Rehabilitation Strategies. (August 2004)
Rahul Jain, B.E., Maharaja Sayajirao University, India
Chair of Advisory Committee: Dr. Dan G. Zollinger

Concrete pavements are facing rapid deterioration due to the increasing high traffic volumes. Maintenance, rehabilitation and reconstruction (MRR) have become major activities for all the state highway agencies. Due to shortage of available funding and continuous aging of pavements, many state highway agencies are now seeking cost-effective MRR strategies. This has led a need to develop a systematic and comprehensive decision process for selecting the optimum MRR strategy that considers pavement, traffic and construction issues.

This research is an effort to help the state highway agencies select the maintenance, rehabilitation and reconstruction strategy for concrete pavements. The research identifies feasibility, suitability and acceptability criteria that every MRR strategy should meet. The rehabilitation strategies satisfying these criteria are then weighed in decision process to determine the optimum rehabilitation strategy. Research also focuses on developing recommendations for statewide methods for rehabilitating jointed concrete pavements so as to minimize reflective cracking.

Data was collected from relevant project case studies to assess and improve the framework for decision process. Further research will be required to enhance the selection process.

DEDICATION

In memory of
My maternal grandparents
My paternal grandfather
My uncle
And my cousin

ACKNOWLEDGEMENTS

I would like to thank all the individuals that supported me during all the phases in my research: Dr. Dan Zollinger, my advisor and Mr. Tom Scullion, principal investigator on the project. I am particularly thankful to Dr. Zollinger for the effort and time he spent with me on the research and his guidance and support on Thesis. I would like to extend my gratitude to my committee members, Dr. Dallas Little and Dr. Larry Ringer for their interest and constructive criticism. Finally, I appreciate my parents for giving me the opportunity to study at Texas A&M.

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INTRODUCTION

GENERAL

Due to combination of increasing volumes of traffic, varying climatic conditions and aging of pavements, many highway facilities are experiencing rapid deterioration. Hence, many concrete pavements may be in poor condition. The total investment involved in improving the deteriorated highway conditions has reached staggering figures, approximately in trillions of dollars. Due to shortage of available funding and increasing revenues, repair and rehabilitation of concrete pavements have become major activities for all state highway agencies.

Since last four decades, Texas Department of Transportation (TxDOT) has been assessing both conventional and non-conventional methods of rehabilitating jointed concrete pavements. But due to inadequate monitoring of pavements, it is difficult to assess the value of each approach. In recent years, asphalt overlays have been increasingly used to rehabilitate the severely distressed Portland Cement Concrete Pavements (PCCP). But reflective cracking is the major concern associated with asphalt overlays. During past two decades, slab-fracturing technique like rubblization, before overlaying has been widely used as means of retarding reflective crack formation in asphalt overlays. Recent research has emphasized the development of pavement-related aspects in the development of repair and rehabilitation strategy selection criteria. But on whole, there has been limited progress made in developing specific strategy selection criteria for concrete pavements with respect to rubblization.

This thesis follows the style and format of the *Journal of Construction Engineering and Management,* ASCE.

OBJECTIVE

The primary objective of this research is to append the guidelines that can be used by state highway agencies to select the appropriate strategies for maintenance, rehabilitation and reconstruction of concrete pavements. Previous researches have identified feasibility, suitability and acceptability criteria that every repair and rehabilitation strategy should meet. This research appends additional criteria to the established criteria. The treatment alternatives satisfying these criteria are then weighed in decision process to determine the optimum rehabilitation strategy. Research also evaluates existing techniques, especially rubblization, to minimize occurrence of reflective cracking in asphalt overlays laid on concrete pavements. Different case studies on rubblization have been analyzed along with evaluation of similar treatments on different areas in various states to assess their relative performance and identify the lessons to be learnt for the future projects

RESEARCH APPROACH

The research approach has been accomplished in four specific tasks, each of which is described below.

1. Literature Review

The research focused on reviewing literature on evaluating pavement distresses to understand mechanism behind the formation of various distresses in the concrete pavement and the way they affect the pavement performance. The research also performed literature review on existing rehabilitation treatments used to minimize reflective cracking in asphalt overlaid concrete pavements. Rubblization along with various existing stress relief layer techniques have been evaluated as a part of literature review. Chapter II presents a summary of the literature review performed on principal pavement distresses and various rehabilitation treatments used to minimize reflective cracking.

2. **Review of Case Studies**

This chapter mainly focuses on the review and analysis of case studies relevant to purpose of this analysis. The review approach first gives brief description of the type of original section and its pre-construction details. It then presents detailed description of the construction activities carried out on the section and also depicts the final description of the section. Finally it summarizes the conclusions learnt from the use of rubblization. The information provided by this review will help us understand how rubblization technique works, its effectiveness and significance and a decision framework that would justify its application.

3. **Append Selection Criteria for Concrete Pavement Rehabilitation**

One of the principal objectives of this research is to modify guidelines that can be used by state highway agencies for maintenance, repair and reconstruction of concrete pavements. Previous researches have identified feasibility, suitability and acceptability criteria, as described in Chapter IV that every pavement repair and rehabilitation treatment should meet. This research appends additional criteria to the previously established criteria. The rehabilitation strategies satisfying these criteria are then weighed in decision process to determine the optimum rehabilitation strategy.

The research went through a specific sequence of steps that are mentioned in the research approach. Research first focused on reviewing literature on evaluating pavement distresses to comprehend mechanism of distress formation in concrete pavements. It also focused on literature review of various techniques used to minimize the occurrence of reflective cracking in the asphalt overlaid concrete pavements. Chapter II presents a summary of literature review on principal pavement distresses and techniques used to prevent reflective cracking. Research also focused on review of case studies relevant to rubblization. Three past projects were mainly reviewed to evaluate the performance of rubblization technique. Chapter III presents a detailed summary of review, analysis and conclusions learnt from these case studies. Finally research focused on appending the existing guidelines for selecting Maintenance, Rehabilitation and Reconstruction (MRR) developed by previous research. Chapter IV elaborates the additional criteria developed as part of acceptability considerations that will help state highway agencies select appropriate MRR strategies.

CHAPTER II
LITERATURE REVIEW

This chapter first focuses on a thorough literature review of some of the principal distresses in the concrete pavements to understand the mechanism behind the formation of distresses in the pavement and the way they affect pavement performance. The second part of the chapter lists the various pavement repair and rehabilitation treatments and presents literature review of rehabilitation techniques used to minimize reflective cracking in the concrete pavements.

CONCRETE PAVEMENT PERFORMANCE

Prior to selecting a pavement rehabilitation strategy, it is critically important to establish what are the predominant pavement distresses and their causes (Maher and Ostiguy 2002). Hence an evaluation of pavement distress is required so as to understand the mechanism behind the formation of the distresses in the pavement and the way they affect the pavement performance. This would help understand the selection of MRR treatments for repairing each pavement distress. Table 2.1 classifies prominent distresses according to their respective type that affect the performance of the concrete pavements.

TABLE 2.1. Concrete Pavement Distress

Distress Type	Functional	Structural	Environmental	Traffic	Material
Durability "D" Cracking					√
Blowups			√		
Corner Breaks				√	
Punchouts		√			
Joint Seal Damage					√
Transverse Cracking		√			
Scaling	√				
Longitudinal Cracking		√			
Spalling		√			
Faulting			√		
Alkali Silica Reactivity					√
Pumping				√	

The evaluation of mechanism of pavement distress is necessary because the lack of structural and functional aspects of pavement can lead to occurrence of pavement distresses that eventually would affect the pavement performance. Following is the description of some of the principal distresses occurring in the jointed concrete pavements.

Spalling

Spalling of cracks and joints is the cracking, breaking or chipping of the slab edges within 2 ft (0.6m) if the joint or crack (Huang et. al 2004). Joint spalling is a construction related distress in the rigid pavement that mainly affects the structural integrity of the concrete slab. The horizontal delaminations in the concrete slab result from early shrinkage of the concrete spread out under various conditions and give rise to spalls. Spalling does not extend vertically through out the entire slab thickness but extends to intersect the joint at an angle.

The primary factors causing the occurrence of spalling include early age characteristic of concrete mixtures, early sawing, improper load transfer at the joints, incompressible material in joints or cracks, alkali-aggregate reaction, D cracking and honey combing in the concrete at joints. Spalling mainly affects the smoothness and ride quality of the pavement. Partial depth and Full depth repairs are used to address spalling, depending upon the level of severity. Figure 2.1 (LTPP Distress Identification Manual, FHWA 2003) shows typical spalling failure in rigid pavements.

FIG. 2.1. Spalling Distress (LTPP Distress Identification Manual, FHWA 2003)

“D” Cracking

“D” cracking is the deterioration of the pavement due to freeze-thaw action in coarse aggregates. It is a material-related distress seen in the concrete pavements. “D” cracking appears as a series of closely spaced, crescent-shaped, hairline cracks that appear at the concrete surface adjacent to and roughly parallel to joints and cracks – and along the slab edge (Huang et al 2004).

The main factors that cause the development of “D” cracking include size of coarse aggregate, properties of coarse aggregates, availability of the moisture and occurrence of freeze-thaw cycles. “D” cracking significantly reduces the structural integrity of the pavements. Slab reduction techniques with overlays are used to eliminate “D” cracking in the concrete pavement. Figure 2.2 (http://www.fhwa.dot.gov/pavement/full2.htm) shows typical “D” cracking in concrete pavements.

FIG. 2.2. High Severity “D” Cracking (FHWA, 2002)

Joint Seal Damage

Joint seal damage is the loss of adhesion between the seal material and the joint faces, removal or displacement of sealer material. It is a material related distress in the concrete pavements. Sealers are used to fill in the space between the joints so that no debris or dust gets accumulated in the pavement surface. Under the influence of weathering and solar radiation, loss of bond occurs between the sealant and joints and the sealer material gets extruded from the pavement surface. The factors that cause joint seal damage may be the use of inappropriate sealant, improper installation of sealant or aging of the sealant. The treatment for joint seal damage is to remove the existing joint sealant, thoroughly clean the joint surface and reseal the joint with proper sealant. Figure 2.3 shows typical joint seal damage in concrete pavements.

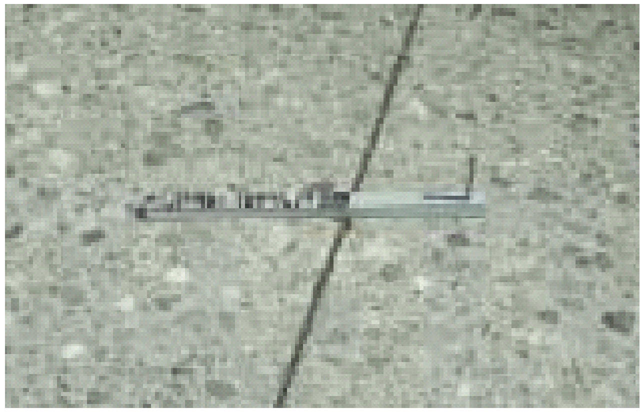

FIG. 2.3. Joint Seal Damage (LTPP Distress Identification Manual, FHWA 2003)

Transverse Cracking

Transverse cracks are randomly oriented cracks that mainly occur at the pavement sections away from joint locations. The cracks are perpendicular to the central line of the pavement. Transverse cracking is the main structural distress occurring in the concrete pavements. The greatest stresses due to wheel loads occur at the outer slab edges, midway between the transverse joints, transverse cracking generally occurs at the mid slab portion of the concrete slab.

Mid slab cracking in concrete pavements occurs due to the fatigue damage under the influence of repeated wheel loading and hence is not much related to the type of coarse aggregate. The various factors that influence transverse fatigue cracking include the magnitude and number of wheel loads, thickness of the concrete slab, stiffness and uniformity of the base, degree of friction between the slab and the base and the degree of load transfer across the joints. There should be adequate reinforcement so that the width of the transverse crack is tightly held and there is proper load transfer across the crack. Figure 2.4 shows typical transverse cracking in concrete pavements.

FIG. 2.4. Transverse Cracking (LTPP Distress Identification Manual, FHWA 2003)

Faulting

Faulting is a discontinuity in the pavement surface across the two adjacent slabs. It occurs at the joints or cracks of the abutting slabs. Faulting is environmental related distress. It is partly caused by the pumping of loose infiltrated materials under the application of heavy wheel loads at the approach slabs and partly due to the loss of support beneath the leave sides of the joints or cracks. The other factors include pumping and curling and warping stresses developed in the pavements and loss of load transfer across the joints. Figure 2.5 shows typical faulting in concrete pavements.

FIG. 2.5. Faulting Distress (LTPP Distress Identification Manual, FHWA 2003)

Faulting significantly reduces the ride quality of concrete pavements. Dowel bar retrofitting is used to repair moderate severity faulting areas. Slab reduction technique and overlay are used to repair high severity faulting areas.

Punchouts

Punchout are localized areas of failures in pavements caused by insufficient structural strength. Edge punchouts are major structural distress of continuously reinforced concrete (CRC) pavements and are characterized by a loss of aggregate interlock at one or two closely spaced, cracks, usually less than 2 ft (1.2 m) apart (Huang et al 2004). Punchouts are mainly caused by shrinkage and close transverse cracking and crack spalling.

The other factors that cause the development of punchouts are number and magnitude of wheel loads, thickness of slabs, stiffness of base, drainage conditions and erosion of support along the slab edge. Also consideration should be given to the aggregate properties such as shape, gradation and wear-out that will govern the aggregate interlock and the crack pattern formation. For low severity punchouts, the affected area is first removed and then patched. For high severity punchouts, slab reduction techniques with overlays are used. Figure 2.6 (http://www.fhwa.dot.gov/pavement/full2.htm) shows typical punchout formation in concrete pavements.

FIG. 2.6. High Severity Punchout (FHWA, 2002)

Scaling

Scaling refers to a network of shallow, fine or hairline cracks that extend only through the upper surface of the concrete pavement (Huang et al 2004). Scaling is caused by improper mix design, over finishing during the construction, improper curing, alkali-aggregate reaction or by a combination of heavy load repetition, loss of foundation support and thermal and moisture gradient stresses. Low severity scaling is not a problem; as it does not affect the structural capacity, ride quality or durability of the pavement. But in high severity scaling, the pieces of concrete pavement surface become loose and the durability and ride quality of the pavement gets affected. Full depth patching and bonded overlays are recommended for high severity scaling. Figure 2.7 shows typical scaling distress in concrete pavements.

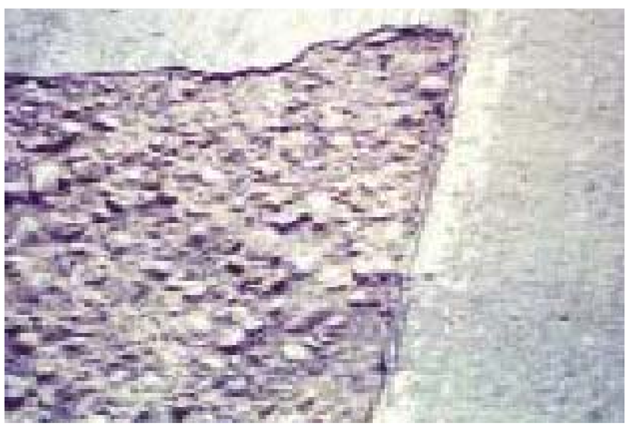

FIG. 2.7. Scaling in JCP (LTPP Distress Identification Manual, FHWA 2003)

Corner Break

Corner break is a traffic load induced distress in the rigid pavement. It is the major structural distress in the concrete pavement. Corner break is a random diagonal crack that occurs at the intersection of the transverse and longitudinal joints. It is a vertical crack that propagates through the entire thickness of the slab. Corner breaks in concrete slab are mainly due to fatigue damage caused by the application of heavy wheel load repetitions at the corner of the slab. Thus corner deflections and stresses are seen in the top surface of the slab that ultimately causes fatigue damage and cracking.

The other factors include the frequency and magnitude of the traffic load, thickness of the concrete slab, stiffness of the concrete slab and base, degree of load transfer at the joints, amount of steel reinforcement, coefficient of thermal expansion of aggregate and curling and warping stresses in the slab. Low severity corner breaks are not a serious concern and hence do not require any treatments. Crack sealing and full-depth patching is performed to repair high severity corner breaks. Slab reduction techniques and overlays are used to repair the pavements that have numerous corner breaks. Figure 2.8 (TM 5-826-6/AFR 93-5, USACE, 1989) shows typical corner break occurrence in the concrete pavements.

FIG. 2.8. Corner Break (TM 5-826-6/AFR 93-5, USACE 1989)

Alkali Silica Reactivity

Alkali Silica Reactivity (A.S.R) is a formation of gel like substance on the surface of concrete pavements. It is a material related distress occurring in the pavement. Fine, closely spaced longitudinal cracks are seen at transverse joints and cracks, joint widths and compressed joint sealant. Reactive aggregates contain silica that chemically reacts with alkalies present in the Portland cement in presence of water to form a gel-like substance. This gel-like substance then absorbs water and swells causing expansion and cracking of the concrete pavements. The other factors responsible for A.S.R include the portion of the reactive aggregates in the mix, alkali content of the cement availability of free water and the particle size of aggregates in the mix. Shoving occurs due to the expansion of concrete pavement with reactive aggregates. Advanced stage of alkali silica reactivity causes blowup distresses in the concrete pavement. Slab reduction techniques and overlay are treatments generally used for repairing this distress. Figure 2.9 (Visual Distress Survey Manual, SDDOT 2003) shows typical A.S.R in the concrete pavements.

FIG. 2.9. Alkali Silica Reactivity (Visual Distress Survey Manual, SDDOT 2003)

Pumping

Pumping is the ejection of water, fines or silt through joints or cracks, caused by the deflection of slab under moving loads (Huang et al 2004). It is a traffic load induced distress in the pavement. As the slab gets deflected, water and fines are pumped vertically upwards through the longitudinal and transverse cracks and joints in the slab. This movement occurs in an opposite direction to the movement of the traffic. Under the influence of heavy wheel loads, the material is forced backwards under the approach corner and then finally it comes upwards through the cracks and joints.

The factors that cause pumping are the poor load transfer across the joints and cracks, erodible base materials, presence of excess water in pavement, warping and curling stresses in the pavements and heavy wheel loads. The treatment for pumping is to install the edge drains to remove the water and fines from the pavement subgrade and seal the cracks and joints. Figure 2.10 shows typical pumping in the concrete pavements.

FIG. 2.10. Pumping Distress (LTPP Distress Identification Manual, FHWA 2003)

Blowups

Blowup is a shattering of concrete pavement slab at joints or working cracks. It is an environmental related distress seen in the concrete pavements. They generally occur in hot weather climates at transverse cracks. Loss of sealant materials from joints causes infiltration of incompressible materials in the joints. During the hot weather period compressive stresses are induced in the slab. Blowups are thus induced due to these horizontal compressive forces. A shattering blowup will crumble the concrete on each sides of the crack. A buckling blowup will raise the pavement elevation by few inches on both sides of the crack.

Concrete pavements having alkali silica reactivity (A.S.R) problems are also potential candidates for blowups. Also pavements having “D” cracking distress are susceptible to blowups. In addition, poor joint sealant conditions and erodible base materials are some of the other factors that can cause blowups in the pavements. The treatment for blowups involves the removal of the affected area and then patching with suitable patch material. Figure 2.11 shows typical blowup in concrete pavements.

FIG. 2.11. Blowup Distress (LTPP Distress Identification Manual, FHWA 2003)

Reflection Cracking

Reflection cracking is the major distress occurring in the asphalt concrete overlays laid on the concrete pavements. Reflection cracks are caused by discontinuities in the underlying layers which propagate through the HMA surface due to movement at crack (Roberts et al 1996). Reflection cracks in AC overlays are caused by combination of thermal and traffic-induced stresses. Expansion and contraction of the PCC pavements results in horizontal movements that produce strains in the AC overlay exceeding its tensile strength (Witczak and Rada 1992). The concentration of strain in joints or cracks might be due to bending or shear stresses that occur under the influence of wheel loads or due to horizontal stresses that occur under the influence of stresses. The load-induced stresses are influenced by the overlay thickness, and thickness and stiffness of the existing pavement layer. Traffic loads and temperature changes causes reflection crack through an AC overlay. Approach using principles of fracture mechanics and beam-on-elastic foundation theory is used in developing mechanistic model for reflective cracking in asphalt concrete overlay (Choi and Lytton 1994). Reflective cracking may also be caused by the existing low temperature cracks in old HMA surface, block cracks induced in old HMA surface and longitudinal cracks in the older surface. Figure 2.12 (Iowa Design Guide, APAI 2003) shows typical formation o74f reflection cracking in the asphalt pavements. Reflective cracking reduces life and serviceability of the pavements.

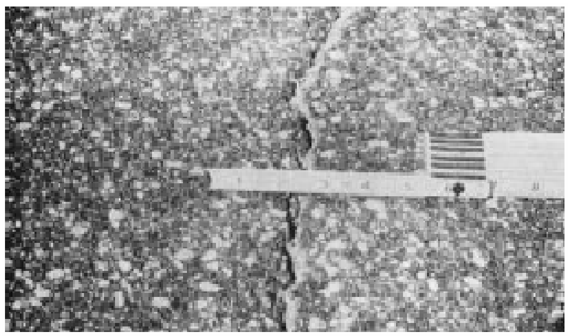

FIG. 2.12. Reflection Cracking (Iowa Design Guide, APAI 2003)

Rutting

Ruts are depressions which occur in the pavement's wheel path as a result of traffic loads (Roberts et al 1996). Pavement might uplift along the sides of the rut. Rutting arises from the permanent deformation occurring in the pavement layers usually caused by consolidation or lateral movements of materials due to heavy traffic loads. Rutting might also occur due to the plastic movement of fines in the asphalt mix in hot weather. Rutting may also occur due to inadequate compaction during construction of new asphalt pavements or due to displacements in the asphalt surface layer. Deficiencies in asphalt mix such as improper aggregate gradation, excessive fines, less binder viscosity; low air void content can cause premature rutting in pavements. Figure 2.13 shows occurrence of rutting in concrete pavements.

FIG. 2.13. Rutting (Iowa Design Guide, APAI 2003)

Ravelling

Ravelling is the progressive disintegration of a HMA layer from the surface downward as a result of the dislodgement of aggregate particles (Roberts et al 1996). Loss of bond between the aggregates and asphalt binder occurs either due to hardening of asphalt, localized areas of segregation in concrete mix or due to low in-place density of the mix. The loss of bond causes separation of asphalt and aggregate that ultimately causes raveling and weathering. Raveling can also be caused by inadequate compaction, construction in colder climates or overheating the mix. Raveling is generally recorded as the square meter of the affected area. However, there are no distress models or techniques available for modeling raveling distress. Raveling and weathering have potential for hydroplaning thereby creating safety hazards for the pavement. Figure 2.14 shows typical raveling in the asphalt pavements.

FIG. 2.14. Raveling (Iowa Design Guide, APAI 2003)

REVIEW OF ALTERNATIVES TO MINIMIZE REFLECTIVE CRACKING

The AASHTO Guide for Design of Pavement Structures ("Guide" 94) uses four categories (4R) which are restoration, rehabilitation, resurfacing and reconstruction. The American Concrete Pavement Association (ACPA) ("Pavement" 1993) describes rehabilitation as restoration, resurfacing and reconstruction. Table 2.2 describes some of the principal MRR alternatives for the concrete pavements.

TABLE 2.2 Repair and Rehabilitation Alternatives for Concrete Pavements

Classification	Function	Treatment Types
Maintenance	Preservation	• Retrofit edge drains
		• Slab undersealing
		• Resealing joints and cracks
		• Thin asphalt concrete overlay
Rehabilitation	Restoration	• Diamond grinding
		• Dowel bar retrofit
		• Partial depth repair
		• Full depth repair
		• Slab grouting
	Resurfacing	• Bonded concrete overlay (BCOL)
		• Unbonded concrete overlay (UCOL)
		• Asphalt concrete overlay (ACOL)
Reconstruction	New Construction	• Remove and replace (R&P)
		• Remove and recycle (R&C)
	Recycle In-place (RIP)	• Crack and Seat (C&S)
		• Rubblize

Maintenance

Maintenance activities are performed for addressing pavement distresses in their initial stages. Maintenance treatments are applied to preserve the functional and structural performance of pavements until such time that either rehabilitation or reconstruction work can be carried out (“Guidelines” 1998). Maintenance activities for rigid pavements mainly include preservation or preventive type of treatments such as retrofit edge drains, slab undersealing, resealing of joints and cracks and load transfer restoration. Due to limited application in the pavements that are already in poor condition due to the existing distresses, application of maintenance activities is justified only when the pavement performance is not adversely affected.

Rehabilitation

Rehabilitation activities are performed when distresses in pavements become severe. Rehabilitation activities can be organized to restore the original pavement condition and also increase the load carrying capacity of the pavement. Desired level of performance, pavement life extension and need for additional structural capacity are some of the main parameters that guide selection of rehabilitation strategy. Rehabilitation activities are mainly grouped as: Pavement rehabilitation activities are mainly classified under three different approaches: Restoration, Resurfacing and Recycling. Restoration and resurfacing provide both short-term and long-term extension of pavement service life

Pavement Restoration

Concrete Pavement Restoration (CPR) is a series of engineered techniques that repairs isolated areas of deterioration in concrete pavement and restore structural and functional integrity of pavements. Diamond grinding, dowel-bar retrofit, partial depth repair, full depth repair and slab grouting are some of the commonly performed concrete pavement restoration activities. Maintenance activities combined with concrete pavement restoration (CPR) techniques mainly find applications in restoring loss of slab support, poor load transfer and improper drainage conditions. Full depth repairs or patching technique is commonly used to restore the rigid pavements that have broken concrete slabs or deteriorated joints. Dowel-bar retrofit is most commonly used technique for restoring load transfer across the undoweled cracks or joints. Diamond grinding technique is used to restore the ride quality in concrete pavements. Diamond grinding in conjunction with full depth repairs technique is used for repairing faulting distresses in rigid pavements. Appropriate and timely CPR extends pavement life by 9-10 years. Some CPR projects have performed for more than 17 years ("Pavement" 1997).

Pavement Resurfacing

Pavement resurfacing includes application of overlays on the rigid pavements so as to enhance their structural capacity and ride quality. Rigid overlays find their application in enhancing the structural capacity of pavements. Flexible overlays are mainly used for improving the functional capacity and ride quality of the pavements. Thin bonded concrete overlays (BCOL) are used to enhance the load carrying capacity of pavements. Hence they enhance the existing life of pavement in terms of both load carrying capacity and useful life in years of the pavement. But they are applicable only when there is sufficient stiffness in pavements.

Reconstruction

Pavement reconstruction is the most expensive form of road rehabilitation and can only be justified where no other recycling options are viable. It involves complete removal of existing granular base and asphalt surfacing (Maher and Ostiguy 2002). Following subgrade preparation, a new pavement is constructed. Reconstruction activities include either recycling the old pavement materials and using it as a base material for the newly laid pavement or entirely constructing the new pavement. The decision for reconstruction of pavement includes consideration of several factors other than for material and pavement considerations. Reconstruction alternatives are applicable when there is lack of insufficient remaining life of pavement, situations where repair and rehabilitation alternatives exceed the reconstruction cost of the pavement and various constraints of projects that limit the use of overlay.

Recycle in-place approach mainly involves application of crack and seat or rubblization techniques, both of which include resurfacing treatments. Full depth asphalt and concrete pavement structures are mainly used as resurfacing treatments for recycling in-place techniques. One of the key benefits of reconstruction is that it improves the drainage condition and enhances the base layer support of the pavement. But doesn't utilize the existing pavement structure as base for the newly laid overlays.

Various rehabilitation techniques aimed at minimizing, reflective cracking have been attempted. They include thick (conventional) overlays with crack relief layers, the saw and seal technique, special overlay and the fractured slab approach. Of these, the technique that has been used increasingly over the last 10 years has been the fractured slab approach (Witczak and Rada 1992). The concept of slab fracturing before overlaying is based on reducing the movement of the cracked or broken slabs beneath the overlay, thereby reducing critical strains in the asphalt (Freeman et al. 2002).

Rubblization

Rubblization involves breaking the existing pavement into pieces having a nominal maximum size of 75 mm or less and above 200 mm or less below any reinforcement before placing a new HMA overlay. This is primarily done to retard the reflective cracking by the total destruction of the concrete slab action. The work further includes the filling of granular material in the depression created by rubblization, compaction and seating (rolling) of the rubblized material, and removal of the waste material. Rubblization is a recycling technique. Several types of distress in concrete pavement require rubblization. The types of distress in concrete pavement that justify the rubblization technique when classified as low, medium or high severity distresses are durability "D" cracking, ASR, spalling and reflection cracking. Figure 2.15 describes typical resonant machine breaker.

FIG. 2.15. Resonant Machine Breaker (RMI 2002)

These distresses cause the concrete pavement to deteriorate and lose its structural integrity. Pavements that are in good condition but have reached the end of structural life are also good candidates for rubblization. Figure 2.16 describes a typical asphalt overlay over rubblized surface. The recognized design procedure for asphalt overlay design of rubblized concrete is AASHTO overlay design of fractured slabs ("Guide" 1994).

FIG. 2.16. Asphalt Overlay over Rubblized Surface (ACPA 1998)

Rubblization is growing in popularity among states in need of rehabilitating their interstate highway pavement without creating the secondary problem of reflection cracking caused when underlying slabs or plates because of temperature changes or loads (Gulick et al. 2002). Bonded Concrete Overlay (BCOL) can also be used to increase the load carrying capacity of rubblized pavement. Application of bonded concrete overlay increases life of existing pavements in terms of both load carrying capacity and functional capacity (McCullough and Uddin 1987). Cracking and Seating (for PCCP) results in lightly breaking the pavement while Breaking and Seating (for JRCP) hits the pavement more closely and results in pieces about a foot across (Wolters et al. 2002). Each of these reconstruction approaches has unique design features and associated construction advantages ("Guidelines" 1998). It reduces the slab to an excellent granular base for the overlay and eliminates all reflective cracking concerns. It can be used when other concrete pavement restoration techniques will not work. The reduction of pavement structural capacity is the primary concern with rehabilitation by rubblization (Freeman et al. 2002). Hence it is not applicable for structurally sound pavements. Many distresses in concrete pavement occur due to poor subgrade conditions. Rubblization destroys the concrete slab's bridging action, exacerbating the problems. This can cause early failures in the asphalt overlay. Hence rubblization may not be advisable for pavements on extremely wet or boggy soils (Cervarich et al. 2001). The soft spots that had been bridged in original concrete pavement show up after rubblization and need to be repaired to ensure adequate pavement performance. It cannot be performed on pavements with delamination type cracks.

However drainage for rubblized pavements is important (Cervarich et al. 2001). Drainage system should be installed to its full operating conditions two weeks minimum prior to rubblization. The rubblized surface should be rolled by a 10 ton, smooth drum vibratory roller. Usually two passes are made to settle the surface fines into the surface cracks producing uniformly smooth surface to pave on. One pass of a water truck over the surface before the first "vibratory" pass helps in preparing a smooth surface for paving. The typical life of a rubblized section with asphalt overlay ranges from 15 to 25 years.

Stress Relief Layer

Stress relief layer is placed between the overlay and the existing pavement. This is mainly done to retard the occurrence of reflection crack in the overlays. Reflection crack is the major distress that arises in newly laid overlays on the older pavement surfaces. It is defined as the propagation of an existing crack pattern, from discontinuities in the old pavement into and through a new overlay. Reflective crack mainly occur in the new overlay surface because of its inability to withstand the shear and tensile stresses created by the movements from the underneath surface of the pavement. The movement is mainly caused by either traffic loading (tire pressure) or due to the thermal loading (expansion and contraction). Face continuity, decrease the structural strength and allow the water to enter the sublayers. Thus the problems that existed in the overlay are extended to the new overlay. Due to this premature failures occur in the new overlays thereby significantly decreasing the service life of the overlay. There are several methods and techniques to control reflective cracking, out of which use of polymer modified mixes and geo-synthetic interlayer have been described in this chapter.

Polymer Modified Asphalt (PMA) Mixes

Polymer refers to a very large unit of molecules made by chemical reaction of many small molecules. The addition of polymer to the asphalt influences the properties of mix depends on the type of polymer system used and the compatibility of the asphalt with this system. Polymer modification causes substantial changes in the stress-strain behavior, creep compliances and non-newtonian flow patterns. The ability of some polymers to elastically recover (elastomers) gives added durability to the asphalt and also enhances its resilient properties. While the polymers that don’t have the characteristics of elastic recovery (plastomers) give higher stability to the asphalt and enhance its stiffness properties. Addition of polymers greatly enhances the resistance of the asphalt mix to rutting, fatigue cracking, and thermal cracking, stripping and temperature susceptibility. Hence they find a huge application in pavements that require extra performing and durable mixes. This chapter mainly focuses on two commonly used polymer modified mixes: Strata® and S.A.M.I

Strata®

Strata® is highly elastic, fine graded, polymer modified mixture. It is an impermeable hot mix reflective crack relief interlayer that is designed to retard the reflective cracking in the asphalt concrete (AC) pavements. Its construction involves first cleaning of the entire pavement surface. Then a tack coat is applied on the pavement surface as per the specified application rates. After this, Strata® interlayer is applied on the pavement surface. The surface is then compacted suitably by using different types of rollers. The appearance of the reflective crack relief interlayer after final rolling is black in color. The surface texture shall be tight. Proper inspection is done to see that flushing has not occurred. The areas showing excessive flushing are removed and replaced.

Strata® mainly finds its application in the jointed concrete pavements that have low severity distress like map cracking, joint deterioration, scaling, spalling, longitudinal cracking and transverse cracking. Concrete pavements with corner cracking and moderate transverse joint spalling distress are also good candidates for Strata® use.

Strata® is very effective in retarding the reflective cracking and also has strong resistance against fatigue cracking. It also acts as a moisture barrier and hence protects the pavement from moisture damage. It can be easily mixed and placed and hence there is minimum traffic disruption. It has sufficient stability to withstand traffic for few days before placing the overlay. But the application is limited to the road surfaces that don't exhibit any structural distresses. It is not applicable to PCC pavements that have high severity D-cracking and patch deterioration. Also it cannot be applied in unfavorable weather conditions. Full depth patch repairs or retrofitted dowel bar techniques are required to be performed in conjunction to repair the pavements that have high severity level distresses like transverse joint faulting, longitudinal cracking and pumping. The reflective crack relief interlayer can only be placed when the minimum air temperature or minimum temperature of the surface on which it is to be laid is 10 C. Pavement should be thoroughly cleaned and dried before the application. The reflective crack relief interlayer cannot be directly placed at the intersection due to potential rutting problems. The typical life of Strata® interlayer technique ranges from ten to fifteen years.

Stress Absorbing Membrane Interlayer

Stress Absorbing Membrane Interlayer (S.A.M.I) is a surface treatment that is very effective for retarding the reflective cracking. S.A.M.I consists of a blend of crumb rubber (20% - 30%) on the total weight of the asphalt-rubber binder. It is first sprayed on the existing surface and then aggregates are spread on this layer of asphalt-rubber binder. Rolling is then done on the surface. Finally a layer of hot mix asphalt is applied on the top of the rolled surface. It is placed on the surface of the road prior to the application of overlay. It is designed to resist the stress and strain of the reflective cracks and delay the propagation of crack

S.A.M.I interlayer mainly finds its application in pavements exhibiting fatigue cracking and low severity level of joint deterioration. S.A.M.I is very effective in absorbing the stress and strain that cause reflective cracking. It gives resistance to aging and hence enhances pavement life. The asphalt-rubber layer is effective in waterproofing and sealing the cracks. It has high aggregate embedment and temperature susceptibility. Due to fast installation there is little disruption in traffic management. But the application is limited to structurally sound pavements. It can’t be applied in unfavorable weather conditions. Figure 2.17 depicts typical layout of S.A.M.I

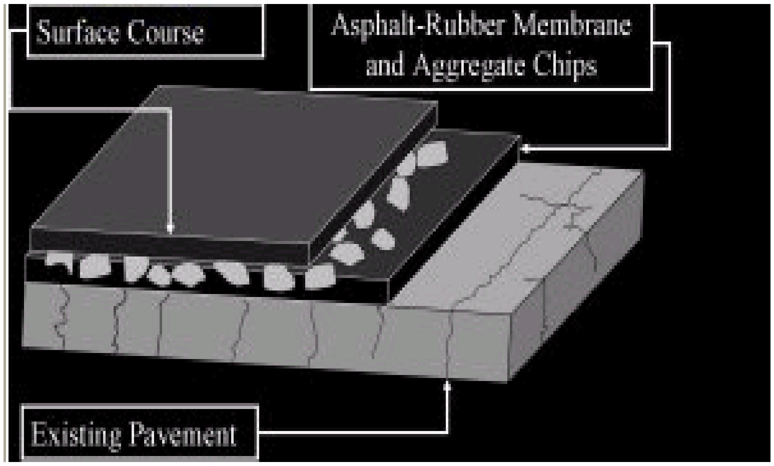

FIG. 2.17. Typical Layout of S.A.M.I (ARTS 2002)

S.A.M.I requires a thoroughly clean and dried surface for its application. It requires a tack coat for adhering to the pavement surface. It is necessary to provide appropriate drainage conditions before the S.A.M.I application. The field performance of S.A.M.I has been excellent. Many projects are performing well after fifteen years of S.A.M.I application.

Geo-synthetic Interlayer

The use of geo-synthetic stress relief interlayer on the pavement surface to retard reflective cracking is also a common practice used by several state department of transportation. The main reason for this is that stress relief layers are very effective in retarding in reflective cracking and the installation process is easy thus causing minimum disruption in traffic. The geo-synthetic layers are mainly of two types: Woven geo-textiles and Non-woven geo-textiles. The geo-synthetic stress relief interlayer retards the occurrence of reflective cracks by absorbing the stress and strain of the crack that arise from the damaged pavement. And as it is embedded in a tack coat it also absorbs the moisture and water that seeps in the pavement through the old cracks and then eventually gets reflected. When reinforced the geo-synthetic interlayer substantially enhances the tensile strength of the pavement. The reinforced geo-synthetic interlayer holds the underlying cracks together and restricts the propagation of the crack along its length. This mainly depends upon axial stiffness of the geo-synthetic interlayer. This chapter focuses on two most commonly used geo-synthetic interlayer: PavePrep® and GlasGrid®.

PavePrep®

PavePrep® is a stress relief, geo-composite interlayer mainly used to retard the reflective cracking in the new asphalt overlays on the older paved surfaces. It consists of high-density mastic laminated between a lightweight non-woven polyester geo-textile and woven polyester geo-textile. It is placed in strips over cracked or spalled concrete sections in pavements. It retards the stresses occurring due to thermal expansion and contraction and hence reduces the tendency of crack reflection through overlay. PavePrep® also acts as a moisture barrier that can cause structural decay of the underlying pavement. Figure 2.18 shows a typical layout of PavePrep® application.

FIG. 2.18. Typical Layout of PavePrep® Surface (PavePrep® Inc 2002)

PavePrep® mainly finds its application in concrete pavements that exhibit low severity joint deterioration, longitudinal cracking and transverse cracking distress. It can also be used to protect expansion joints in concrete pavements from freeze thaw action.

The PavePrep® mastic acts as a stress absorbing membrane and hence retards reflective cracking. It isolates new asphalt overlay from the old surface and hence absorbs the differential movements between the old and new surface. It prevents the moisture entering the new surface and hence checks structural decay of pavement. It retains the thickness even after exposure to vehicular traffic. With easy and fast installation there is minimum traffic disruption. But the application is limited to the pavement surfaces that don't exhibit structural distresses. Storage requires perfect insulation from moisture and rain. It also requires tack coat for adhering to the pavement surface.

The PavePrep® shall be dry prior to the installation. It shall be adhered to the existing surface by applying a tack coat. The non-woven polyester side of the material shall be embedded in the tack coat. The woven polyester side of the material shall be kept exposed to the traffic. PavePrep® shall be only applied when the existing surrounding temperature is at least 50 F or more. PavePrep® installation required adequate drainage conditions in the pavement. The typical life of PavePrep® ranges from 10-15 years.

GlasGrid®

GlasGrid® is a self-adhesive reinforcing mesh used as a stress relief interlayer to retard the reflective cracking in the new asphalt overlays on the older paved surfaces. It consists of high tensile strength fiberglass strands arranged in a grid structure and covered with a polymer coating and a pressure sensitive adhesive. The construction includes first thorough cleaning of the pavement. A layer of tack coat is placed on the surface. And then the grid is placed on the surface by mechanical means. The laid grid surface is then compacted by a suitable roller. GlasGrid® can be applied on asphalt as well as concrete pavements that have cracks due to thermal stresses. It is also useful to repair the pavements that have cracks due to fatigue. It can also be used on cracks caused by uneven settlement, given that crack formation is not due to excessive pavement instability and on the joints in the concrete pavements and for road widening.

GlasGrid® has tremendous strength to weight ratio and hence gives high tensile strength and stiffness. It has high asphalt compatibility and thermal stability. The polymer coating provides reinforced overlays with sufficient adhesion to maintain a good bond between asphalt concrete overlays. It provides high mechanical interlocking within the composite system. Thus asphalt particles can have better compaction, greater bearing capacity and increased load transfer with less deformation. It allows easy and quick installation with minimal traffic disruption. It doesn't require tack coat for adhesion to the pavement surface. But the application is limited to structurally sound pavements only. It is not applicable to the pavements that have high severity pumping distress and slippage or excessive movements. It requires a minimum overlay thickness of 40 mm. Figure 2.19 shows typical layout of GlasGrid® pavement.

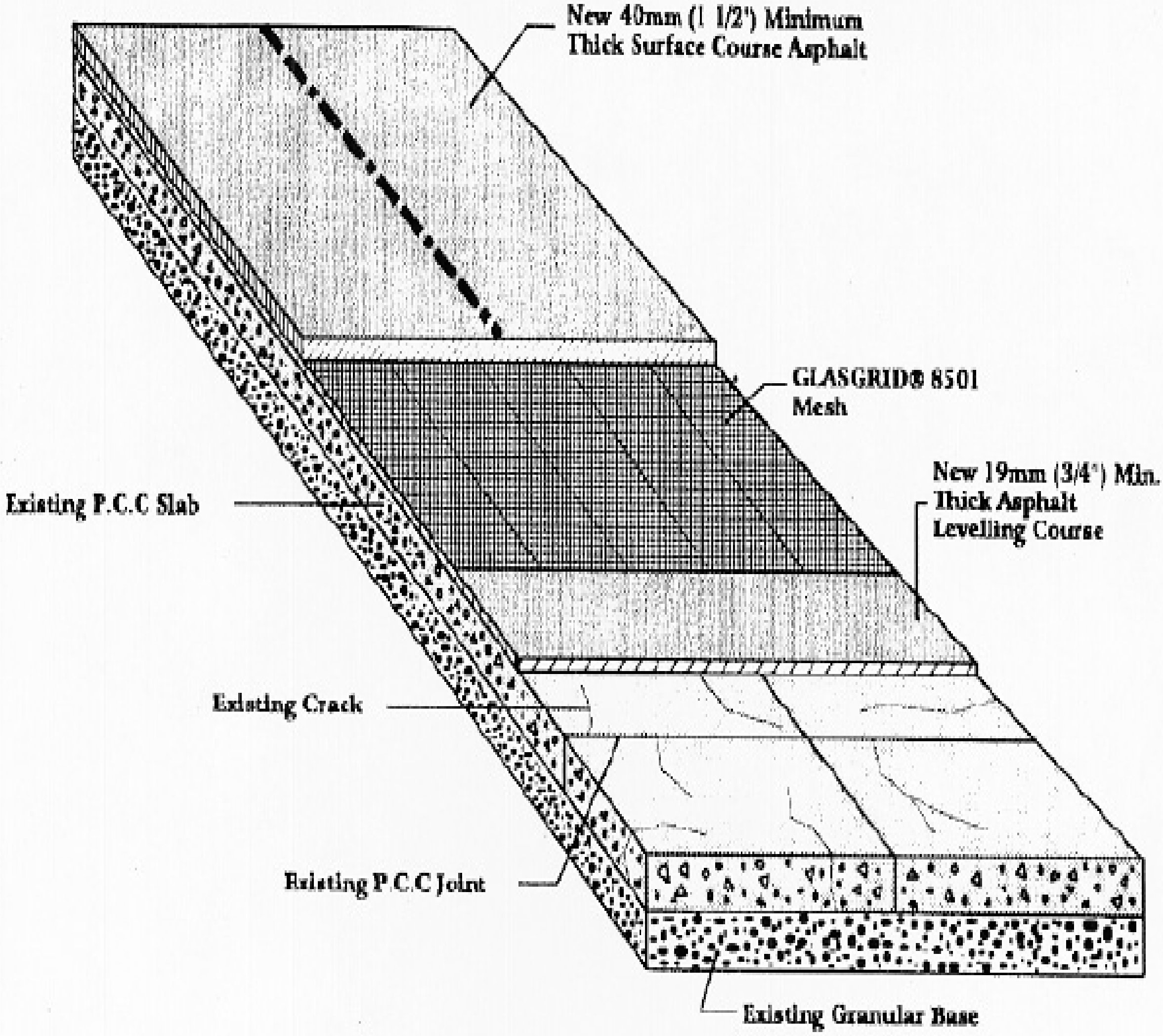

FIG. 2.19. GlasGrid Pavement Layout (GlasGrid Tech Manual 2000)

CHAPTER III
REVIEW OF CASE STUDIES ON RUBBLIZATION

This chapter mainly focuses on the review and analysis of case studies relevant to rubblization technique and its performance. Three rubblization projects were selected for the purpose of this analysis. The first is a break and seat project of US 169, Nowata County, Oklahoma. The other two projects are rubblization projects on US 59, Oklahoma. The review approach first gives brief description of the type of original section and its pre-construction details. It then presents detailed description of the construction activities carried out on the section and also depicts the final description of the section. Finally it summarizes the conclusions learnt from the use of rubblization.
The information provided by this review will help us understand how rubblization technique works, its effectiveness and significance and a decision framework that would justify its application.

BREAKING AND SEATING OF US 169, NOWATA COUNTY

Project Number MAF-193 (45) was the first large-scale rubblizing project undertaken by the Oklahoma Department of Transportation (ODOT). It is located on US 169. Beginning at the junction of SH 28 and US 169, it extends up to north for 10 km (6.2 mi) to the junction of SH 10 at Lenaph (Brewer et al. 1991). The US 169 highway not only carried heavy car traffic as a commuter corridor, but also served a large volume of truck traffic due to the major ports and terminals it connected. From the traffic study done by Research and Development division, 1991, it was reported that US 169 had an Average Daily Traffic (ADT) of 2300 vehicles per day, 15% of which were trucks.

The PCC pavement was built in 1972. It consisted of 200 mm (8 in) thick PCC pavement laid over a 100 mm (4 in) thick fine aggregate bituminous base. The fine aggregate bituminous base was laid on a clayey Subgrade. The pavement was 15 foot jointed slab without any dowels and had 8-foot asphalt concrete shoulders. A preliminary evaluation of US 169 in June 1988 showed several severe joint deterioration problems in the pavement along with cracking. Cracking covered more than 50 percent of the surface area (Brewer et al. 1991). Spalling was seen in majority of joints due to an advanced level of "D" cracking. The spalling joints were 13 mm (0.5 in) to 38 mm (1.5 in) deep and extended 50 mm (2 in) to 76 mm (3 in) on each side. The smoothness profile index of the pavement indicated that the pavement had a very rough riding surface.

The Oklahoma Department of Transportation engineers chose "Break and Seat" technique as the rehabilitation alternative. They mainly chose this method to solve two problems: First was to repair a highly spalled and patched PCC pavement. Second was to reduce reflective cracking in the AC overlay (Brewer et al. 1991). The rehabilitation method involved breaking the pavement with a resonant breaker, seating the pavement with a roller, placing a leveling course on the pavement, placing a reinforcement fabric over leveling course and laying a thick dense graded asphalt mix overlay. The PCC pavement was broken into a rubblized base by a resonant breaker. A rubber-tire roller was used to seat the rubblized pavement. Steel wheel roller was then used to smooth the pavement surface. A leveling course was laid of Type "B" mix was laid on the pavement. Reinforcement fabric was laid on leveling course. Finally 180 mm (7 inch) thick dense graded asphalt mix overlay was placed. Entire project was completed by May 1989.

The second year evaluation showed increased rutting and minor bleeding in wheel path. But the reflective cracking over the joints was eliminated (Brewer et al. 1992). Raveling was identified as the major distress after the five years of construction. The pavement also showed rutting and alligator cracking. From the five-year evaluation, it was found that rubblizing prior to the laying of an asphalt concrete overlay was an effective method in reducing the reflective cracking. Also rubblizing of the PCC pavements was found to be cost effective against the full depth repair and replacement.

Pre-Construction Testing

A preliminary investigation of US 169 in Nowata County was made in June 1988. The investigation showed severe joint deterioration problems along with numerous cracking and small amount of rutting. Spalling was seen in majority of joints due to an advanced stage of “D” cracking in the pavement. “D” cracking in the pavement occurs at joints due to the break down of the aggregate structure under the influence of repeated freeze-thaw cycles. It mainly occurs due to the use of coarse aggregates that are porous in nature. Porous aggregate easily gets water saturated. Disintegration then occurs in these porous aggregate. “D” cracking in US 169 is mainly attributed to the use of Lenaph limestone coarse aggregates. Cracking covered more than fifty percent of the surface area of the pavement and was prominent at the corners. The spalling joints were 13 mm (0.5 inch) to 38 mm (1.5 inch) in depth and extended 50 mm (2 inch) to 76 mm (3 inch) on each side. The profilograph survey indicated that the pavement had a very rough riding surface.

Construction

The construction of US 169 began in August 1988. There were several routine and maintenance patches on the road. So cold milling was done on the pavement to remove all these patches. A two inch thick standard Type “B” leveling course with surface course of Type “B” was used. Shoulders widening was done to allow traffic movement on the other shoulder during the rehabilitation process. The PCC pavement was rubblized by a resonant breaker on September 19, 1988 (Brewer et al. 1991). The resonant breaker was a self propelled PB 4 model manufactured by Resonant Technology Company, Reno, Nevada. Vibrating at a frequency of 45 Hertz with a 2000 lb force, the resonant breaker produced low amplitude of 11/16 inch in a12/12 inch wide foot. The resonant breaker worked 1000-foot lengths in each pass and took 15 passes to finish the 12-foot lane.

The PCC pavement was rubblized into gravel size pieces by the resonant breaker. This caused voids in the rubblized pavement that caused a 0.5-inch increase in grade. Diesel pile hammers were used to rubblize the PCC pavement over concrete box culverts which rubblized PCC pavements into large pieces. These large pieces were then replaced and removed by a stabilized aggregate over the fine aggregate bituminous base.

Seating

Two different rollers mainly did seating of the rubblized PCC pavement: 50 T Rubber tire roller made five passes on the rubblized pavement. This was done to seat the rubblized pavement on to the original fine aggregate bituminous base. 35 T Steel wheel rollers made the sixth pass on the rubblized pavement to smooth the surface.

The pumping and soft spots seen after rolling operation were removed. They were filled with a dense graded asphalt concrete mix and were further compacted by a Vibrating roller. A leveling course of Type "B" insoluble that matched the grade of pavement shoulders was placed on the finished surface by asphalt laydown machine. Then a four-ounce non-woven polypropylene fabric was placed on the leveling course. This fabric acted as a moisture seal and as a reinforcement layer for the pavement. A subsequent binder course of Type "A" was then placed on the shoulders and mainline of the pavement. Vertical prefabricated edge drains were cut in the shoulders at the original pavement edge. Vermeer trencher was used to install 12 inch "Hydraway" prefabricated composite edge drains from Monsanto Corporation. The southbound lane of U.S 169 had 500 feet of Advanced Drainage Systems Composite Drain and 4500 feet of 305 mm (14 inch) Hydraway edge drains.

On December 1988, three coats of SS-1 Type emulsion were sprayed on the binder course. A thick tack coat was applied to seal off any voids in the surface of the Type "A" binder course. The construction of the road was stopped during the winter season. The construction of the road was restarted on April 17th, 1989. There were no signs of base failures or any visible cracking during the three-month period. However settlement had been observed that had caused some noticeable dips in the pavement. Hence a Type "B" mix was placed on the settled areas. A two inch thick surface course of Type "B" mix was placed on the mainline and the shoulders. The surface course on mainline was laid by two asphalt laying down machine running side by side for a hot joint at the center line.

The mainline was laid for the first two days and shoulders were laid for the next two days. This alternate sequence was followed until the entire work was completed. Entire construction of the road was completed in May 1989.

Evaluation

Several field investigations were conducted to evaluate the pavement performance over a period of five years. The final five-year evaluation was completed in August 1994. Following five field investigations were mainly conducted for evaluating the pavement performance evaluation: Pavement condition rating survey, Crack mapping of the pavement, Profilograph measurements of pavement, Deflection data from non-destructive Benkelman Beam Deflection Test and Measuring Rut depth in pavement.

Pavement Condition Rating Survey

The condition rating survey was carried out for the entire pavement section was "Average". After the five years of construction, Raveling was identified as the main distress arising at the road surface (Brewer et al. 1995). The other main distresses identified on the roadway were Cracking and Rutting. Cracking wasn't seen during the first year pavement evaluation. But it gradually increased by each year. The longitudinal cracking that appeared in pavement was fatigue cracking (Brewer et al. 1995). The pavement also showed an increased level of rutting after the first year evaluation. This could have occurred due to degradation of asphalt concrete mix. The average rut depth of the pavement was approximately 5 mm. Patching was performed on the pavement. Approximately 670 square meters of patching was done on the 9.9 km of the pavement. This was only about 0.5 percent of the road surface and thus was an undersized amount. However there was no reflection cracking at the joints of the concrete pavement. So it seemed, as rubblization was effective in stopping reflective cracking in the pavement.

Crack Mapping

Crack mapping was done on selected pavement sections over a period of five years. This was mainly done to evaluate the cracks occurring on the pavement. Three 60 meter (200 feet) sections were randomly chosen for crack mapping.

Section I

Section I is located 0.3 km north of SH 28. Approximately 10% of the entire section showed cracking (Wilson et al. 1995). This longitudinal cracking was fatigue-cracking occurring in the wheel path. Figure 3.1 depicts graphical representation of cracking in Section I, U.S 169.

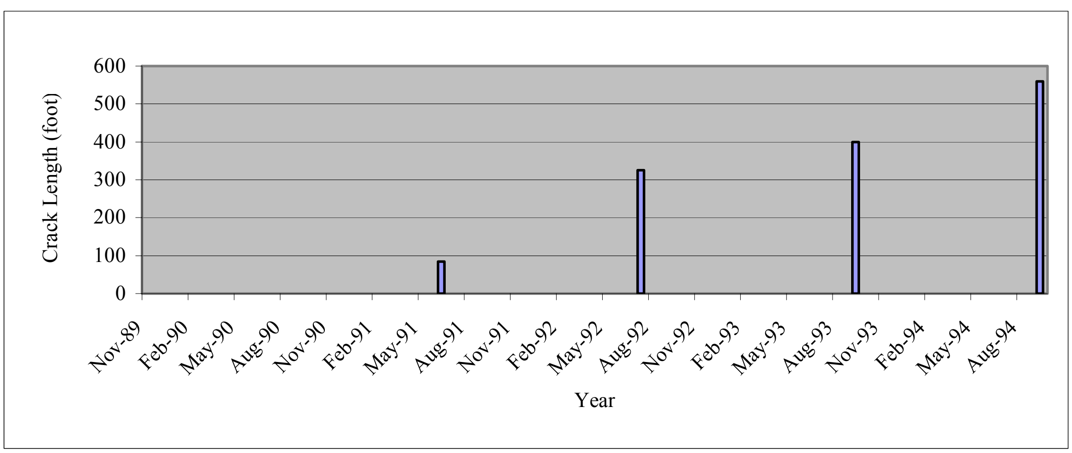

FIG. 3.1. Cracking in Section I, US 169

Section II

Section II is located 7.2 km north of SH 28. Approximately 30 percent of the entire section showed longitudinal and random cracking (Brewer et al. 1995). There was considerable cracking in Section II. Figure 3.2 depicts graphical representation of cracking in Section II, U.S 169.

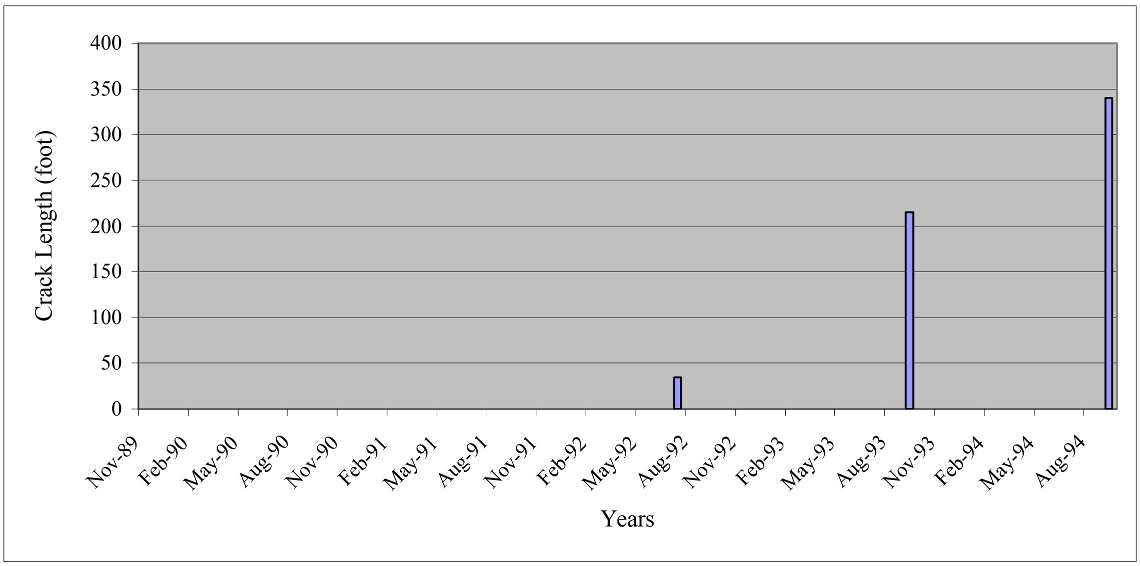

FIG. 3.2. Cracking in Section II, US 169

Section III

Section III is located 9.0 km north of SH 28. It showed more than 50% of cracking (Brewer et al. 1995). The cracking was fatigue cracking and rutting. This section showed considerable cracking. Figure 3.3 depicts graphical representation of cracking in Section III, U.S 169.

Profilograph Measurements

The profilograph survey was done to determine the smoothness profile index of the roadway. The profilograph was run on the outside wheel path in both the lanes. The measurement intervals were at every 0.3 km. The average profile index was considered for each lane. The average smoothness profile index of northbound lane was 145 mm per kilometer. The southbound lane had average smoothness profile index of 154 mm per kilometer. Thus southbound lane had higher readings than the northbound lane. Both the lanes had smooth riding surface.

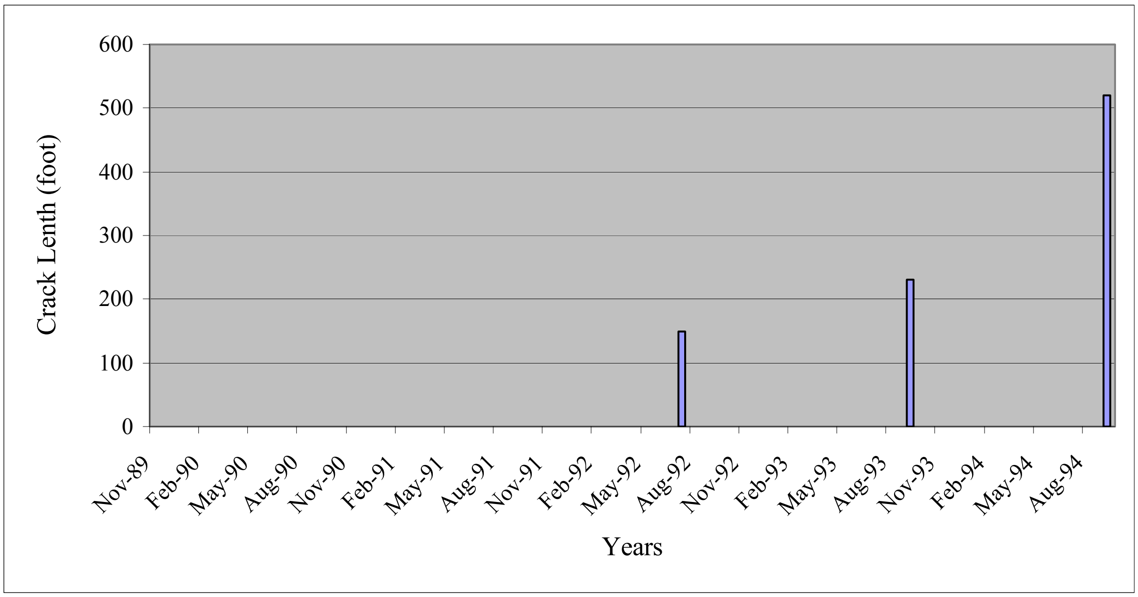

FIG. 3.3. Cracking in Section III, US 169

Deflection Data from Benkelman Beam Test

Benkelman beam deflection testing was done to evaluate the structural strength of the pavement. Thirty-two locations were tested in the northbound locations. Five out of these thirty-two tested locations gave low readings of pavement supporting capability. This indicated the need of an overlay. Similarly thirty-two locations were tested in the southbound lanes. Eleven out of the thirty-two locations gave low readings of pavement supporting capability, hence indicating a need of an overlay. Figure 3.4 shows Benkelman Beam deflection readings for the northbound lanes. Figure 3.5 shows Benkelman Beam deflection readings for the southbound lanes.

Measuring Rut Depth

The rut depth measurements were taken of the pavement section. The average rut depth for the pavement section was 5 mm. 12% of pavement didn't show rutting. Approximately 20% of the pavement had rut depths less than 8 mm. As the average rut depth was about 5 mm that is within limits rutting was not a significant problem.

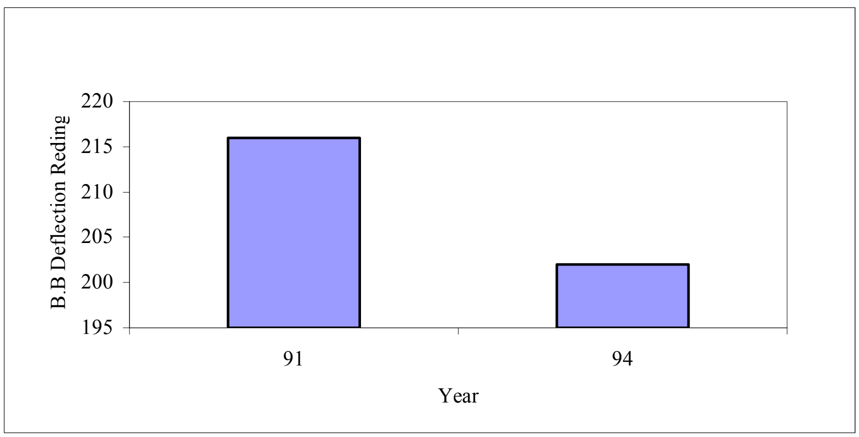

FIG. 3.4. Benkelman Beam Deflection Readings for Northbound Lanes

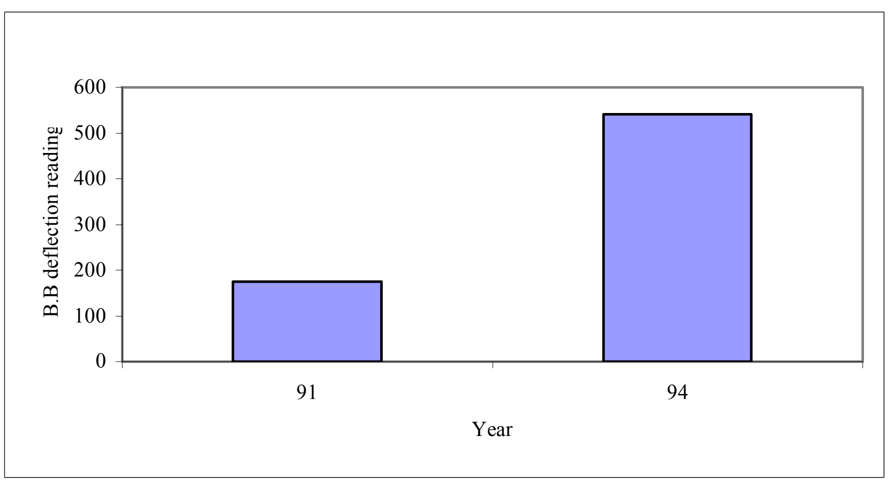

FIG. 3.5. Benkelman Beam Deflection Readings for Southbound Lanes

Conclusion

The “Break and Seat” operation on US 169 was carried out in an efficient way with no major problems. No cracking was visible during the first year of evaluation and the pavement condition rating was “Good”. But the second year evaluation did show an increase in cracking and bleeding spots on the pavement. Deflection testing indicated that structural capacity of the pavement was intact. Cracking gradually increased thereafter year by year. The result was that some of the rehabilitated sections showed higher levels of pavement deterioration in the five-year period. This arises questions regarding the performance of rubblizing.

After five-year evaluation, raveling was identified as the principal distress on the pavement. Raveling is mainly attributed to the aggregates used in asphalt concrete mix. Engineers from Oklahoma Department of Transportation (ODOT) carried the Absorption test on coarse aggregate samples from the overlay. It was found that coarse aggregates had 3.3 percent absorption. This indicated high void content in the aggregates due to which raveling might have occurred.

Cracking and rutting were also seen in the five-year evaluation survey. The longitudinal cracking was the fatigue cracking seen in the wheel paths of the pavement. Fatigue cracking in pavements is mainly caused by the action of repetitive strains on the pavement layer that causes reduction in the tensile strength of the layer material that ultimately leads to failure of the layer materials. Fatigue cracking in US 169 might have occurred due to aggregate asphalt characteristics used in asphaltic concrete overlay. The rut depth measurements were taken of the pavement section. The average rut depth for the pavement section was 5 mm. As the average rut depth was about 5 mm that is within limits rutting was not a significant problem.

From the five-year evaluation, it could be said that the raveling and cracking distresses that occurred in the pavement might have been due to aggregate or asphalt and hence were not related to rubblizing. The only maintenance activity performed on the pavement was patching. Approximately 670 square meters of patching was done on the 9.9 km of the pavement. This was about 0.5 percent of the surface and hence was insignificant.

Deflection data of the pavement was obtained from Benkelman Beam deflection testing and the Falling Weight Deflectometer testing. Benkelman Beam deflection readings were taken at every 1000 feet in the outside wheel path of the roadway. The Benkelman Beam deflection readings didn't indicate any loss in the structural capacity of the pavement over the period of five years. Also the Falling Weight Deflectometer readings indicated that structural capacity of the pavement was intact.

The profilograph survey was run on the outside wheel path of both the lanes to determine the smoothness profile index of the roadway. The southbound lane had an average of 154 mm of roughness per kilometer (9.8 in/mi). The, northbound lane had an average 145 mm of roughness per kilometer (9.2 in/mi). This indicated that the pavement had a smooth riding surface.

Also no reflective cracking was observed in the pavement after the five years of construction. From this it could be concluded that rubblizing of PCC pavement prior to laying asphalt concrete overlay was an effective solution against full depth replacement in reducing the reflective cracking in the pavement. Rubblization is also a cost-effective solution. Hence it was concluded that for Portland Cement Concrete (PCC) pavements having "D" cracking distress were good candidates for rubblization.

Rubblization needs adequate subbase strength and minimum joint seal damage so as to have best performance. The pre-construction monitoring of the section indicated that section had good subbase strength. Hence rubblization was the preferred alternative as section could bear the pressure of rubblizing operation. The section had severe joint deterioration problems, which were corrected prior to the rubblizing operation. Hence the section gave better performance in future. Chapter 4 describes the guidelines on subbase strength and subgrade drainage required for adequate performance of rubblization.

SECTION 400607, US 59, OKLAHOMA

Original Section

The original pavement consisted of 9.1inch JRCP on a 100 mm (4 inch) sand base laid on 200 mm (8 inch) subbase layer made of soil aggregate mixture, predominantly clay, resting on silty clay (SC) subgrade.

Final Section

Pre-construction Monitoring

The preconstruction meeting for this project was held on January 28, 1992 (Daleiden et al. 1995). Preconstruction monitoring included the following measurements before the start of rubblization. This was done to assess the pavement conditions prior to the application of rehabilitation treatment.

Pavement Surface Distress

From the distress surveys conducted on October 11, 1991 and July 28th , 1992, moderate faulting, low severity spalling and corner breaks were the main distresses identified on the pavement section.

Surface Profile

Rod and level measurements of the pavement section were taken prior to rubblization. Also, longitudinal profile of the section was obtained from SHRP's high-speed profilometer on January 14th, 1992.

Structural Capacity

Structural Capacity of the pavement was evaluated from deflection measurements using a SHRP Falling Weight Deflectometer from January 28th – February 6th, 1992.

Materials Sampling and Testing

Oklahoma D.O.T, in coordination with SHRP SRCO, conducted pre-construction sampling on June 3rd, 1992. The sampling operation involved extraction of 100 mm (4 inch) and 150 mm (6 inch) diameter cores, probes and three test pits of 2 meter by 1.2 meter size to a depth of 3.5 meter below the top of the untreated sub-grade.

Construction

The rubblization on section 400607 began on the afternoon of July 27th, 1992. The concrete pavement was rubblized with RMI Breaker. Operating at a frequency of 44 beats per second it made 20 passes per lane. The concrete pieces on the surface were about 50 mm to 75 mm in size and those below the steel were closer to 150 mm in size. The outside lanes were rubblized on July 27th, 1992. Inside lanes were rubblized on July 28th, 1992. Two sets of deflection measurements were taken before start of seating operation.
A 39 Ton pneumatic roller was used to seat the rubblized concrete. Two passes were made over each section (Daleiden et al. 1995). After the seating operation, the entire pavement was water blasted and then air blasted. This was done to remove the dust and fines that could inhibit the bonding of the asphalt concrete AC to the surface. Deflection measurements were taken on the newly seated pavement section before the application of the overlay.

Paving operation was done on the section using a SS-1H tack coat with 50% dilution rate (1 part diluents to 1 part asphalt). Starting on July 29th, 1992 the paving operation was completed by August 7th, 1992. A Caterpillar 2000 Drum Mixer plant (Daleiden et. al

1995) was used to lay the hot mix asphalt concrete overlay on the section. A first lift of Type B mix AC overlay was placed on July 29th, 1992. The second lift of Type B mix was placed on August 7th, 1992. Following three different rollers were used to compact the overlay: A 10 Ton Hyster steel wheeled vibratory roller was used as a break down roller that made two passes over the section, a 12 Ton Bomag pneumatic roller was used as an intermediate roller that made five passes over the section and a 13.5 Ton Hyster steel wheeled static roller was used as a final roller that made two passes over the section.

The installation of edge drains for sub drainage started on July 30th, 1992. The main purpose of the sub drainage installation was to remove the free water from the drainage layers. The Advanedge pipe system was used for the sub drainage. The pipe system had corrugated plastic rectangular channels encased within the filter fabric. High modulus geo-textile wrap was used as a primary filter. It was closely placed to the slab. The top of the channel was placed 1inch below the PCC slab surface and the horizontal distance of the pipe from the outer edge of the pavement is 3 inch. Laterals were then cut through the shoulders to dispose the drainage of the system through the shoulders. The lateral drains were placed by August 3rd, 1992. All the traffic had been detoured during rubblizing operation and installation of edge drains for the sub-drainage system. The road was opened to the traffic after the completion of the above operations.

Post Construction Monitoring of the Section

The post construction monitoring, similar to the pre construction monitoring of the section was initiated after the completion of the above operations. It was mainly done to assess the effect of various operations on the performance of the road section.

Pavement Surface Distress

The manual distress data obtained on November 5th, 1992 did not show any signs of distress on the road sections. The following table describes the various types of cracking that occurred in Section 400607.

Surface Profile

Longitudinal Profile of the section was obtained from SHRP's high-speed profilometer March 16th, 1993. Rod and level measurements were also taken on the section. Table 3.1 describes the various types of cracking that occurred in Section 400607.

Sampling and Testing of Materials

Oklahoma D.O.T conducted sampling & testing of materials on August 31st, 1992. Precise sampling was done to extract asphalt cores without any splitting of the samples from the section overlaid with HMAC. The obtained samples & the samples obtained from the pre construction sampling were sent to the laboratory for testing.

Structural Capacity

The structural capacity of the pavement was evaluated from the deflection measurements using a SHRP Falling Weight Deflectometer (F.W.D) on April 1993.

TABLE 3.1. Types of Cracking in Section 400607
(Breaking and Seating of U.S 59 Oklahoma, 1992)

Time in Years	Alligator Cracking (square feet)	Transverse Cracking (linear feet)	Longitudinal Cracking (linear feet)	Alligator + Patching (linear feet)
11/5/1992	0	0	0	0
3/30/1994	0	8	0	0
11/2/1994	0	14	0	0
5/22/1997	161	213	1025	161
11/17/1998	322	292	1060	322
9/14/2000	418	12	0	5395
8/3/2001	0	0	0	5581
9/17/2001	0	0	0	0

Back-calculation of Layer Moduli Using MODULUS 6.0

MODULUS 6.0 was used to calculate the elastic modulus of different pavement layers. Table 3.2 shows the elastic modulus of different pavement layers for Section 400607.

TABLE 3.2 Layer Moduli for Section 400607

Year	Temperature (F)	E_{HMA}	$E_{CONCRETE}$	E_{BASE}	$E_{SUBGRADE}$
Pre-construction	75	--------	5220.5	18.6	19.5
1993	93.2	786	47.6	67.3	16.3
1996	17.6	1814.5	143.1	32.4	18.5
1998	62.6	978.4	83.4	98.1	18.5

Conclusion

Rubblization operation on Section 400607, US 59 was carried out in an efficient way with no major problems. No cracking was visible during the first year of evaluation and the pavement condition rating was “Good”. The next two-year evaluation showed the occurrence of slight transverse cracking in the pavement section. Deflection testing indicated that the structural capacity of the pavement was intact. Cracking gradually increased thereafter year by year. As a result some of the rehabilitated sections showed higher levels of pavement deterioration in the five-year period. This arises questions regarding the performance of rubblization.

The six-year evaluation period identified rutting and longitudinal cracking as principal distresses in the concrete pavement. Patching was the only maintenance activity performed on the pavement. No reflective cracking was found in the pavements during the evaluation period. This suggests that rubblizing of PCC pavements prior to laying asphalt concrete overlay is an effective solution for minimizing reflecting.

SECTION 400608, US 59, OKLAHOMA

Original Section

The original test section consisted of 9.1 inch JRCP on a 100 mm (4 inch) sand base layer laid on 200 mm (8 inch) subbase layer of soil aggregate mixture, predominantly clay, resting on silty clay (SC) Subgrade.

Final Section

The construction of final section was done in the following three stages:

Pre-construction Monitoring of the Section

Pre-construction monitoring included the following measurements before the start of rubblization. This was done to assess the pavement conditions prior to the application of rehabilitation treatment.

Pavement Surface Distress

From the distress surveys conducted on October 11th, 1991 and July 28th, 1992, moderate faulting, low severity spalling and corner break were the main distresses identified on the pavement section.

Surface Profile

Rod and level measurements of the pavement section were taken prior to rubblization. Also, longitudinal profile of the section was obtained from SHRP's high-speed profilometer on January 14th, 1992.

Structural Capacity

Structural Capacity of the pavement was evaluated from deflection measurements using a SHRP Falling Weight Deflectometer from January 28th – February 6th, 1992.

Materials Sampling and Testing

Oklahoma D.O.T, in coordination with SHRP SRCO, conducted pre-construction sampling on June 3rd, 1992. Sampling operation mainly involved extraction of cores, auger probes and three test pits below the top of the untreated subgrade.

Construction

The rubblization on section 400608 began on the afternoon of July 27th, 1992. The concrete pavement was rubblized with a RMI (Resonant Frequency) breaker. Operating on frequency of 44 beats per second it made 20 passes per lane. The concrete pieces on the surface were about 50 mm to 75 mm in size and those below the steel were closer to 150 mm in size. The outside lanes were rubblized on July 27th, 1992. The inside lanes were rubblized on July 28th, 1992. Two sets of deflection measurements were taken before the start of the seating operation.

A 39 Ton pneumatic roller was used to seat the newly rubblized pavement. The roller made seven passes per lane. After the seating operation, entire pavement was water blasted and then air blasted. This was done to remove the dust and fines that could inhibit the bonding of AC to the surface. Deflection measurements were taken of the newly seated pavement section before the application of the overlay.

Caterpillar 2000 Drum Mixer plant was used for laying the hot mix ac overlay on the section. A first lift of Type A mix AC overlay was placed on July 29th, 1992. The second lift of Type A mix AC overlay was placed on August 3rd, 1992. The paving operation was done on the section using a SS-1H tack coat with 50% dilution rate (1 part diluents to 1 part asphalt). The paving operation was started on July 29th, 1992 and was completed on August 3rd, 1992. Then surface friction course of Type B mix was placed on August 7th, 1992. Three different rollers were used to compact the overlay: 10 Ton Hyster steel wheeled vibratory roller was used as Break Down Roller that made two passes over the section, 12 Ton Bomag pneumatic roller was used as intermediate roller that made five passes over the section and 3.5 Ton Hyster steel wheeled static roller was used as a final roller that made two passes over the section.

The installation of edge drains for sub-drainage system started on July 30th, 1992. The main purpose of the sub-drainage installation was to remove free water from drainage layers. The Advanedge pipe system was used for the sub drainage. It had corrugated plastic rectangular channels encased within filter fabric. High modulus geo-textile wrap was used as a Primary filter. It was closely placed to the slab. The top of the channel was placed 25 mm below the PCC slab surface and the horizontal distance of the pipe from the outer edge of the pavement is 75 mm. Laterals were then cut through the shoulders to dispose the drainage of the system through the shoulders.

Post-construction Monitoring of the Section

The post-construction monitoring, similar to the pre-construction monitoring of the section was initiated after the completion of the above operations. It was mainly done to assess the effect of various operations on the performance of the road section.

Pavement Surface Distress

The Manual distress data obtained on November 5, 1992 did not show any signs of distress on the road sections.

Surface Profile

Longitudinal Profile of the section was obtained from SHRP's high-speed profilometer on March 16th, 1993. Rod and level measurements were also taken on the section. From the profile obtained, it can be seen that the ride quality of the pavement has increased after the application of these processes. Table 3.3 describes the various types of cracking that occurred in Section 400608.

TABLE 3.3. Types of Cracking in Section 400608 (Breaking and Seating of U.S 59 Oklahoma, 1992)

Time in Years	Alligator Cracking (square feet)	Transverse Cracking (linear feet)	Longitudinal Cracking (linear feet)	Alligator + Patching (linear feet)
11/5/1992	0	0	0	0
3/30/1994	0	0	0	0
11/2/1994	0	0	0	0
5/22/1997	0	49	1004	0
11/17/1998	11	92	1001	11
9/14/2000	231	277	1021	231
8/3/2001	474	437	1027	474

Structural Capacity

The structural capacity of the pavement was evaluated from the deflection measurements using a SHRP Falling Weight Deflectometer (F.W.D) on April 1993.

Sampling & Testing of Materials

Oklahoma D.O.T conducted sampling & testing of materials on August 31st, 1992. In spite of precise sampling, only one complete core could be obtained for the AC overlay placed on Section 400608. The splitting of the samples during coring indicated that the aggregates were not bonded well in the bottom part of the overlay.

Back-calculation of Layer Moduli using MODULUS 6.0

MODULUS 6.0 was used to calculate the elastic modulus of the different pavement layers. Table 3.4 shows the elastic modulus of different pavement layers for Section 400608.

TABLE 3.4. Layer Moduli for Section 400608

Year	Temperature (F)	E_{HMA}	$E_{CONCRETE}$	E_{BASE}	$E_{SUBGRADE}$
Pre-construction	75	---	5114	1837	19.7
1993	95	626.8	168.1	25.8	19.6
1996	19.4	2418.6	196.8	30.1	23.0
1998	66.2	830.3	298.5	40.7	25.6

Conclusion

Rubblization operation on Section 400608, US 59 was carried out in an efficient way with no major problems. No cracking was visible during the first year of evaluation and the pavement condition rating was “Good”. The next five-year evaluation showed the occurrence of longitudinal and transverse cracking in the pavement section. Deflection testing indicated that the structural capacity of the pavement was intact. Cracking gradually increased thereafter year by year. As a result some of the rehabilitated sections showed higher levels of pavement deterioration in the five-year period. This arises questions regarding the performance of rubblization.

The ten-year evaluation period identified rutting and longitudinal cracking as principal distresses in the concrete pavement. No reflective cracking was found in the pavements during the evaluation period. This shows that rubblized PCC test sections were outperforming other test sections (Daleiden et al. 1995) and rubblizing pavements prior to laying asphalt concrete overlay is an effective solution for minimizing reflecting.

CHAPTER IV
SELECTION CRITERIA FOR PAVEMENT REHABILITATION

INTRODUCTION

The primary objective of pavement rehabilitation treatment selection is to determine the optimum rehabilitation strategy for the distressed pavements or pavements that are likely to deteriorate to an unacceptable level. The procedures involved in this process include first developing list of feasible rehabilitation treatments as per the predetermined set of selection criteria. This list of feasible rehabilitation treatments is then checked for the acceptability and suitability criteria to identify the optimum rehabilitation alternative.

All the pavement repair treatments are broadly classified into following three categories: Maintenance, Rehabilitation and Reconstruction (MRR). Maintenance treatments for pavements generally include “repair as needed” type fixes that incorporate corrective type measures to restore the pavement to its functional or structural condition. Rehabilitation activities include repair treatments that significantly extend the life of an existing pavement such as through resurfacing and restoration. Resurfacing refers to providing a new overlay or wearing course to improve the safety, rideability and skid resistance of the concrete pavement. It may also enhance the structural capacity of the concrete pavement. Restoration refers to application of several techniques that restore structural condition and rideability of the distressed pavement to an acceptable level. All the maintenance and preventive type treatments focus on life extension and hence, should be applied at the appropriate stage or pavement condition. To ensure that the optimum MRR strategy will be selected the full scope of rehabilitation strategies need to be evaluated:

- Preventive treatments address problems created due to current levels of distress.
- If corrective treatments are used, but only if they are needed to achieve the target rehabilitation life. Treatment options may include overlays and meeting the target

rehabilitation life, not achieving the longest life extension possible. Structural overlay options (both HMA and PCC overlays) may do this at different levels of cost.

- Reconstruction can often be the best rehabilitation option for extensively deteriorated pavements.

Table 4.1 presents a detailed classification of pavement rehabilitation strategies into following general categories: Maintenance, Rehabilitation and Reconstruction.

TABLE 4.1. Pavement Rehabilitation Types (Zollinger et al. 2001)

Classification	Function	Treatment Types
Maintenance	Preventive or Preservative	• Clean drains
		• Retrofit edge drains
		• Reseal joints and cracks
		• Thin asphalt concrete overlay
		• Slab undersealing
Rehabilitation	Corrective or Restoration	• Diamond grind
		• Restore load transfer
		• Retrofit edge support
		• Partial depth repair
		• Full depth repair
		• Slab jacking
	Resurfacing	• Unbonded concrete overlay
		• Bonded concrete overlay
		• Asphalt concrete overlay
	Recycle In-place (RIP)	• Crack and Seat
		• Rubblize
Reconstruction	New Construction	• Remove and replace
		• Remove and replace

Developing an optimum rehabilitation strategy for the pavement repair treatments involves a process of passing checks on selected rehabilitation strategies for feasibility, acceptability, and suitability criteria. Feasibility is a basic requirement that all possible alternatives must satisfy. Feasibility involves minimum conditions with respect to service life, traffic levels, and exposure to certain climatic conditions that should be satisfied by every selected strategy. The utility of the selected set of feasible alternatives is further checked with respect to acceptability criteria involving structural and functional adequacy. Once technically feasible alternatives are checked for acceptability, the passing alternatives are checked for suitability criteria that include considerations for cost effectiveness and important non-pavement related factors weighed in the decision process to determine the optimum solution. The criteria associated with the checks for feasibility, acceptability, and suitability (FAS) is largely formulated based upon user input.

This chapter presents decision framework for identifying possible MRR strategies based on engineering criteria. A summary of needed input factors affecting the feasibility, acceptability and suitability analysis of MRR strategies is first discussed. This is followed by a discussion the process for feasible strategy selection is presented, in the form of a decision tree. The output from this process is a set of candidate MRR strategies (which are possible, sound engineering solutions) that can be evaluated for acceptability and suitability. As routing maintenance is mainly designated by policy, it has not been considered in the strategy development. The checks identified for feasibility, suitability and acceptability will constitute the set of guidelines evolved for identifying possible MRR alternatives for a variety of conditions.

FACTORS AFFECTING MRR STRATEGY DEVELOPMENT

Table 4.2 lists key criteria or relative to the feasibility of an MRR strategy, along with various attributes associated with them and a brief description of how they affect rehabilitation strategy development

TABLE 4.2. Factors affecting MRR Strategy Development (Zollinger et al. 2001)

Factor	Attribute	Significance to Rehabilitation Selection
Pavement condition	▪ Distress type ▪ Distress severity ▪ Distress extent	▪ Identifies feasibility of various rehabilitation treatments
Cause of distress	▪ Structural related ▪ Construction related ▪ Materials related ▪ Functional related	▪ Identifies the type of problem: structural, functional, or materials-related (MRD) ▪ Determines applicability of various rehabilitation treatments ▪ Identifies the need for structural enhancement
Traffic	▪ Design ESALs (expected traffic volume and axle weights) - Low - Medium - High	▪ Affects rate of deterioration
Climate	▪ LTPP climatic regions (temperature, moisture, and free-thaw cycles) - Wet-Freeze - Wet-Nonfreeze - Dry-Freeze - Dry-Nonfreeze	▪ Affects rate of deterioration
Rehabilitation life	▪ Rehabilitation life - Short-term (<5 years) - Intermediate-term (5 to 10 years) - Long-term (>10 years)	▪ Determines the level of rehabilitation effort needed

In general, these criteria are mainly related to the following:

- Current pavement condition
- Causes of pavement deterioration.
- Rate of deterioration
- Rehabilitation objective

The level of rehabilitation needed depends on the desired rehabilitation life and rate of deterioration, which, in turn, depend on traffic and climate. For example, full-depth repair is a feasible MRR treatment for transverse cracks in JPCP, but the decision on what severity distresses to repair depend on the required rehabilitation life, expected traffic, and prevailing climatic conditions. If the expected rehabilitation life is only 5 years where the rehabilitation objective is to repair only the distresses that cause excessive roughness, then the cracks that need to be repaired are:

- Low traffic (Traffic < 1000 trucks per day)
    - Dry-nonfreezing climate—none
    - Other climate—high-severity only
- Medium and heavy traffic (Traffic> 1000 trucks per day)
    - Dry-nonfreezing climate—high-severity only
    - Other climate—medium- and high-severity

The cause of distress affects long-term effectiveness of MRR treatments since it is related to the basis mechanism of the failure process. For example, if a pavement is structurally deficient, continued cracking of the original pavement will be a problem, even if full-depth repairs are made to address existing cracks. Although considered in greater detail under acceptability criteria, those projects may require a structural enhancement (e.g., HMA or PCC overlay), depending on the desired rehabilitation life. If severe material-related problems (such as D-cracking or alkali silica reactivity (ASR)) exist, repair treatments will likely only provide temporary relief from roughness caused by the material-related distresses (MRD).

A feasibility decision tree would be based on the assumption that not all pavement distresses require repairing as long as they do not pose a functional problem or contribute to accelerated pavement deterioration. For example, transverse cracks in JPCP have minimal effect on serviceability while they are low-severity, but they can cause significant roughness when the cracks develop faulting and hence, all cracks those are likely to deteriorate, for a given traffic level, into medium- or high-severity within the desired period would be addressed. For a short rehabilitation life in a favorable climatic condition and under light traffic, a possible MRR strategy is to simply diamond grind to remove existing roughness. Under heavy traffic in an adverse climate, full-depth repair of all existing cracks is more appropriate.

The above example illustrates a rehabilitation strategy in which repairs are made as needed, based on functional criteria. In this approach, only those distresses that cause excessive roughness and those that are likely to deteriorate into an unacceptable condition within a given period are selected for repair. The level of rehabilitation efforts needed is determined by site factors (traffic level and climate). In cases dictated by the input conditions, structural enhancements (such as a hot mix asphalt (HMA) overlay) may also be needed to prevent future development of distresses that may require a corrective action within the period of time the rehabilitation should remain serviceable.

BASIC INPUT INFORMATION

Various types of information, from both field and historical records, are needed to evaluate MRR strategies for PCC pavement repair. The required information includes the following inputs:

Pavement Geometry Data

Pavement geometry data includes thickness of PCC pavement, type of base and subbase, design of joints and shoulders, type of drainage and type of subgrade. This information is needed to determine the remaining life of the existing pavement structure. These data are also needed to back calculate material properties using falling-weight deflectometer testing data. This information can be obtained from drawings and construction records.

Pavement Condition Data

Pavement condition data that includes description of the severity and quantity of all pavement distresses is needed in determining possible MRR strategy. Detailed visual distress survey is done to obtain the current pavement condition data needed to evaluate MRR needs. The data also includes functional performance data such as ride, surface friction, and noise data that help assess the functional requirements of the pavement.

Material Properties

Information regarding the properties of materials such as strength and modulus of PCC base and or subbase, subgrade are required to assess remaining life of the existing pavement structure. The material properties can be determined by conducting destructive (e.g., core testing) and nondestructive testing (FWD testing).

Traffic Data

Traffic data is needed to determine remaining life of the existing pavement structure and predict future performance of pavement. Traffic data is required in terms of vehicle load groups but is also characterized as the number of equivalent single axles (ESALs) in most design procedures. Although the procedure for determining ESALs from traffic data is provided in the *AASHTO Guide for Design of Pavement Structures* (AASHTO 1993), some adjustment should be made relative to the mechanism of failure a particular treatment is addressing.

Environmental Data

Environmental data in terms of existing pavement temperature, precipitation and freeze-thaw cycles is also needed to assess the existing pavement structure. The environmental data can also be important in identifying the cause of distress. The LTPP classification is manly used to characterize various regions as per the climatic conditions.

FEASIBILITY APPROACH

The basic feasibility approach for screening possible MRR strategies is to “package” MRR treatments based on the existing pavement, traffic and climatic conditions. It also considers the rehabilitation objectives required to address all existing pavement deficiencies. The main objective of this step is to screen all MRR strategies in terms of possible combinations of distresses and eliminate those that are not suitable for further analysis (Anderson et al. 2002). The following three general categories of rehabilitation lives are defined for MRR strategy selection:

- Short-term rehabilitation life target rehabilitation life less than 5 years. The rehabilitation objective in this category generally involves corrective measures for minimal repair in order to bring the pavement condition up to the minimum acceptable level until another comprehensive rehabilitation or reconstruction technique can be performed.

- Intermediate-term rehabilitation with target rehabilitation life from 5 to 10 years. This category typically includes repair and restoration techniques to restore serviceability of deteriorated pavements and extend the pavement life until the next rehabilitation or reconstruction technique is performed. They may also include preventive measures (e.g., an overlay or retrofitted dowel bars) to achieve the desired performance life.
- Long-term rehabilitation with target rehabilitation life of 10 or more years. This category includes comprehensive rehabilitation strategies performed on the deteriorated pavements in order to provide long-term service. Generally overlay or other preventive measures are needed to address structural inadequacies of the original pavement, thereby avoiding the need for additional repairs within the rehabilitation period due to the continued deterioration of the original pavement.

Feasible MRR Strategy Selection Flowcharts

Based on the concepts discussed above, comprehensive flowcharts have been developed for identifying 'packages' of possible MRR alternatives. The flowcharts identify possible MRR strategies for different rehabilitation conditions based on:

- Type of failure
- Rehabilitation life
- Site conditions

The cause of distress (type of failure) is an important factor that determines the appropriate course of MRR action. The possible MRR strategy selection flowcharts separate projects based on the type of failure as follows:

- Material-related problems—pavements with D-cracking, ASR, or other MRD. Foundation movement problems—pavements with swelling soil or frost heaving problems.
- Functional problems—pavements with excessive noise, excessive roughness, or inadequate surface friction.
- Structural problems—pavements with structural distresses.

The feasibility flowcharts separate projects based on failure mode to address the different MRR needs for different types of pavement conditions. For material-related distresses, it is useful to determine an MRD rating of the pavements.

Due to limited effectiveness, CPR is not recommended on pavements having MRD rating more than 3. If MRD rating is 1 or 2, CPR can be effective, but the rehabilitation life will be governed by the remaining life of the material. Therefore, the following options are provided for pavements with different degrees of MRD:

- MRD rating of 1 represents mild material distress condition. For short- and intermediate-term solution refer to the MRR strategies provided in the flowcharts for either short- or intermediate-term designs. For long-term solution unbonded overlay, HMA overlay with slab fracturing, or reconstruction is recommended.
- MRD rating of 2 represents moderate material distress condition. For short-term solution, refer to MRR strategies provided in the flowchart for short-term design. There is no provision for intermediate-term solution. For long-term solution unbonded overlay, HMA overlay with slab fracturing, or reconstruction is recommended.
- MRD rating of 3 represents severe material distress condition. There are two options. First option is to leave the pavement to deteriorate until reconstruction or an overlay is needed. Second option is to consider unbonded overlay, HMA overlay with slab fracturing or reconstruction as the available alternative.

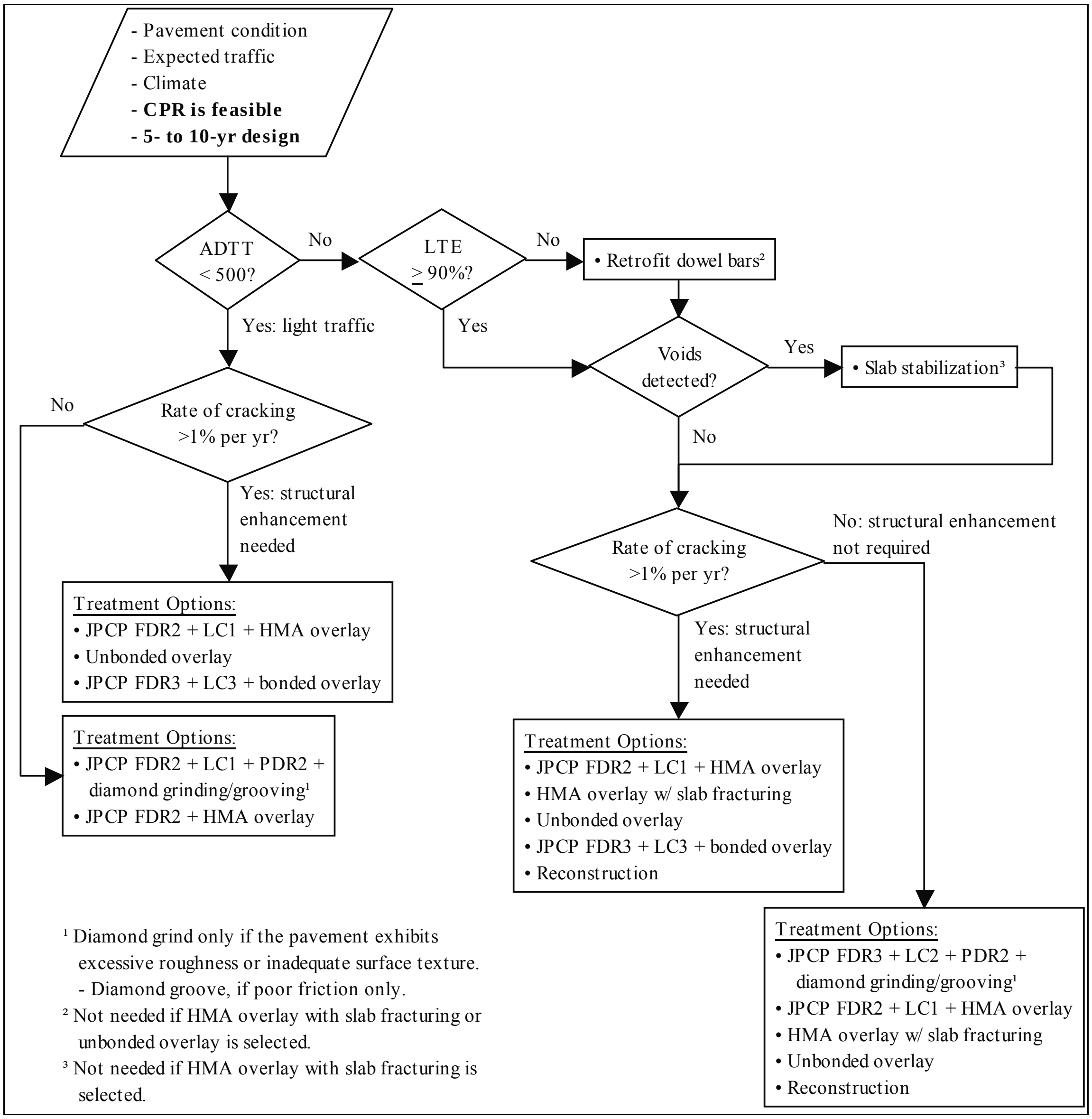

FIG. 4.1. Examples of Possible MRR Strategy Selection Flowchart for JPCP, Intermediate-term Design (Zollinger et al. 2001)

Figure 4.1 describes typical flowchart for JPCP (intermediate-term rehabilitation). Figure shows MRR alternatives with different scopes provided for each case.

- JPCP FDR3 + LC2 + PDR2 + diamond grinding/grooving. This is "repair as needed" CPR alternative, which includes minimum CPR needed to ensure a 5 to 10 year rehabilitation life. JPCP FDR3 includes full-depth repair of all shattered slabs, all transverse cracks (with the option to retrofit dowel bars or tie bars across low-severity cracks), medium and high-severity longitudinal cracks in the wheelpath. LC2 includes cross-stitching or retrofitting with tie bars all medium-severity longitudinal cracks (both wheelpath and non-wheelpath). PDR2 includes partial-depth repair of medium- and high-severity partial-depth joint spalling. Diamond grinding or grooving is specified if the pavement exhibits excessive roughness or has inadequate surface texture. Diamond grooving is recommended, if poor friction is the only functional problem.
- JPCP FDR2 + LC1 + HMA overlay. This option is the more comprehensive level of rehabilitation. JPCP FDR2 includes full-depth repair of all shattered slabs, medium- and high-severity transverse cracks, high-severity longitudinal cracks in the wheelpath, and medium- and high-severity corner breaks. LC1 includes cross-stitching or retrofitting with tie bars medium-severity longitudinal cracks in the wheelpath.
- HMA overlay with slab fracturing. Similar to the previous option, this option is mainly used to minimize reflection cracking and hence provide more reliable performance. In general, a thicker overlay is required after slab fracturing. Not all PCC pavements may be good candidates for slab fracturing (Ksaibati et al. 1999). Unbonded overlay option can be used to provide a rehabilitation life that is similar to a new PCC pavement. Reconstruction option can be considered when the pavement has reached the end of its structural and functional life.

The alternative MRR strategies listed above are a set of possible combinations of MRR treatments (in increasing order extensiveness) that will provide adequate functional and structural performance for a minimum of 5 to 10 years under the specified site conditions.

If the existing pavement is structurally deficient, a structural enhancement may be needed to prevent future deterioration. In the selection flowcharts, the rate of slab cracking (JPCP) and rate of crack deterioration (JRCP and CRCP) are used to identify pavement sections with structural deficiency. This criterion is adequate for the purposes of identifying possible MRR strategies. The actual impact of structural enhancements on LCC is considered in the assessment of acceptability, where pavement performance is used for determining the preferred MRR strategy.

ACCEPTABILITY CONSIDERATIONS

For further enhancing the process of rehabilitation strategy development relative to acceptability criteria, a decision analysis tool configured for condition assessment is required that is capable of accounting for the improvement in pavement condition afforded by a variety of MRR techniques. This capability is founded in a rating scheme of the functional and structural capacity contributions relative to each treatment type. The rating scheme proposed for this purpose is based upon utility theory. Utility theory is used in the case to synthesize and account for a variety of factors that play a role to varying degrees that affect the best choice of MRR techniques. In a study carried out for the FHWA, (Ledbetter et al. 1979) developed a systems methodology for decision analysis in the selection of pavement rehabilitation strategies and treatments for asphalt and concrete pavements using utility theory that accounted for factors associated with cost, performance, safety, and energy usage. Utility theory facilitates a way to compare dissimilar things (i.e. apples and oranges) based on the following concepts:

- Value. It is the worth attached to an object or a service.
- Utility. It is capability of a practice or measure to satisfy a particular need or provide a desirable result.

Utility theory provides rationality basis in acceptability criteria for pavement rehabilitation. Hence acceptability criteria represent set of prerequisite preferences associated with the end results of a combination of MRR treatments, which will hopefully lead to the best combination maximizing a variety of objectives (such as life extension, overall pavement condition, or performance). Furthermore, such preferences can be rated in terms of their relative utility (typically on a scale from 0 to 1) or ability to satisfy the acceptability criteria (Ledbetter et al. 1979).

Table 4.3 also shows the acceptability criteria for the variety of treatment combinations derived from the feasibility criteria. It basically outlines the criteria behind the further consideration of MRR strategy combinations that would be appropriate for a given set of project conditions. The rating process relates to four possible types of strategies to be adopted, namely whether to conduct routine maintenance, to conduct CPR, to consider the use of different types of overlay, or to undertake total reconstruction. Table 4.3 provides a list of the pavement condition attributes that are considered to be important in the acceptability criteria in terms of structural, functional, and material-related distress. This table suggests how various rating indices relative to these distress types are formulated and taken into account. These rating indices are noted at the bottom of Table 4.3 where some of them are made up of subcomponents. These indices are configured as follows:

- Structural Condition (SC)
    - Cracking (HMAC, JRCP, and JPCP)
    - Existing patch density
    - Punchouts (CRCP)
- Functional Condition (FC)
    - Profile
    - Skid
    - Noise
    - Subgrade Drainage
    - Subbase Strength

- Profile (P)
    - Ride quality
    - Existing spall density
    - Rutting (HMAC)
    - Faulting (JRCP and JPCP)
- Overall (OC)
    - Structural
    - Functional

Table 4.3 also shows how both structural and functional condition ratings are formulated in terms of their relative components. Individual distress types determine the structural condition rating of the pavement. Whereas attributes like ride, skid, and noise quality determine the functional condition rating of the pavement. An overall condition rating for the given pavement segment can be derived from the weighted rating of each decision attribute noted in the table. This weighted-value is compared to the decision criteria, as part of determining which strategy type should be selected. Since it is expected that the rating criteria will dictate the life-cycle economics behind any optimized MRR strategy, both the weights and the criteria limits assigned to each decision attribute are user-defined inputs into the process, however, suggested weights for each of these indices are provided in appendix D.

TABLE 4.3 Treatment Combination/Strategy Selection Decision Indices (Zollinger et al. 2001)

Strategy Type	Decision Attribute	Weighted Attribute Component**	Suggested Decision Criteria Limits (% of scaled value)
To Conduct Routine Maintenance (*Cost driven solution*)	Structural Condition (SC)	Distress Type Distress Level Remaining Life (RL)	If SC Rating > 50% If RL Rating > 50%
	Functional Condition (FC)	Ride Profile Skid Resistance Tire Noise+ Subgrade Drainage Subbase Strength	If FC Rating > 25%
	MRD Condition (DC)+	D-Cracking ASR Dowel/Steel Corrosion	Provided in table 4.4
To Conduct Repair (CPR) (*Engineering driven solution*)	Structural Condition (SC)	Distress Type Distress Level Remaining Life	If SC Rating < 50% If RL Rating < 50%
	Functional Condition (FC)	Ride Profile Skid Resistance Tire Noise Subgrade Drainage Subbase Strength	If FC Rating < 50%
	MRD Condition (DC)	ASR Steel Corrosion	Provided in table 4.4
To Use Overlay	Suitability for Overlay	Life Extension (LE)+	LE Rating > 80% (Jointed) LE Rating > 70% (CRC) LE Rating > 50% (HMA)
To Reconstruction	Suitability for Reconstruction	Lane Capacity (LC)+ Remaining Life (RL)+ Life Extension (LE)	LC Rating < 50% RL Rating < 50% LE Rating < 25%

*****Weighting criteria is suggested in appendix G.*

+ Ratings are user defined

Structural Condition (SC) Rating = $U_{SC} = \sum W_{D_i} U_{D_i} + W_{Pat} U_{Pat}$

- U_{D_i} *is the rating for individual distress types and levels (*U_{SC} *does not include faulting (F) distress for jointed pavements but does include thermal and reflective cracking for HMAC pavements)*
- W_i *represents the weight given to each rating*

Functional Condition (FC) Rating = $U_{FC} = W_P U_P + W_{SKID} U_{SKID} + W_{NOISE} U_{NOISE} + W_{DR} U_{DR} + W_{SS} U_{SS}$

- *Profile (P) Rating =* $U_P = W_{RIDE} U_{RIDE} + W_F U_F + W_{Spall} U_{Spall} + W_{Rut} U_{Rut}$
- *Drainage (D) Rating =* $U_{DR} = W_K U_K + W_{CS} U_{CS} + W_{Depth} U_{Depth} + W_{JS} U_{JS}$
- *Permeability Rating =* U_K
- *Cross Slope Rating =* U_{CS}
- *Depth of Flow Line Rating =* D_{epth}
- *Joint Seal Damage Rating =* U_{JS}
- *Skid Rating =* U_{SKID}
- *Noise Rating =* U_{NOISE}
- *Patching Rating =* U_{Pat}
- *Rutting Rating =* U_{Rut}
- *Spalling Rating =* U_{Spall}
- *Strength Rating =* U_{SS}

Overall Condition (OC) Rating = $U_{OC} = W_{SC} U_{SC} + W_{FC} U_{FC}$

Nonetheless, in this manner, the flexibility referred to previously is provided in the determination of acceptability. Each relevant index is assigned a utility, which is a measure of preference represented by a value between 0 and 1 but can be scaled to any convenient magnitude. It is determined by either the user or by use of a utility curve. All rating indices relevant to the selection of a given strategy are given in table 4.3. Utility curves for the structural distress types are provided in appendix E while those for functional distress types are provided in appendix F.

To facilitate computerized application of the utility curves, a general form of a utility distress curve is expressed by the following equation (Stampley et al. 1995):

$$U_i = 1 - \alpha e^{-(\frac{\rho}{L})^{\beta}} \tag{4.1}$$

where:

U	=	utility value
i	=	a pavement distress type
$\forall, \exists, \Delta$	=	utility curve coefficients (1.15, 1, and 20, respectively for the rating curve shown in figure 4.2)
L	=	level of distress or remaining life

Given the type of distress, a utility rating curve may be configured to represent cracking, spalling, faulting, or any of the distresses that change with time or traffic. Since the rating varies with the severity of the distress, the utility approach can be integrated into the acceptability criteria because the severity of distress is a factor. In this manner, each distress type can be compared and weighted on a scale that is universal throughout the comparison process.

The acceptability criteria limits listed in table 4.3 serve as threshold or trigger values that govern the type of strategy that should be pursued. Two other indices listed in table 4.3

are remaining life (RL) and life extension (LE). RL is a before treatment rating that represents the user's satisfaction that the pavement section's service level will be maintained for a certain projected time. LE is an after treatment rating that represents the user's satisfaction that the treated pavement section's service level will be maintained for a certain projected time. Both of these ratings are user-defined meaning that utility curves are not provided for them. In other words, the user determines these ratings relative to the service life the user wishes to obtain with either of the treated conditions. In terms of criteria levels noted in Table 4.3, it is pointed out, that corrective type treatments tend to fall under the routine maintenance category since these types of treatments do little to change the rate at which the pavement is deteriorating, which actually dictates how the criteria levels for the SC and RL indices are configured. Elaborating further, it can be argued that CPR encompasses treatments that do change the rate of deterioration, which reverses the orientation of the SC and RL indices relative to their respective criteria limits.

Additional Criteria for Rubblization

Rubblization is among the principal pavement treatment alternatives to minimize the occurrence of reflective cracking in asphalt overlays laid on concrete pavements. Selection of rubblization alternative requires evaluation of structural, functional and material related distress condition of the pavement, as described above. This task describes two additional criteria, strength and drainage conditions of soils, which are added to the functional criteria that would aid the evaluation of rubblization as an acceptable alternative.

Subbase Strength

Rubblization is an effective rehabilitation alternative for the deteriorated Portland cement concrete pavements especially to minimize reflective cracking. However, rubblization involves breaking of concrete pavements by pavement breaker and hence creates the demand of subbase that is strong enough to handle the breaking operation. The Dynamic Cone Penetrometer (DCP) test is the most preferred technique for measuring the in-situ strength of subbase soils in rigid pavements. DCP test index has been used as the parameter for evaluating the strength characteristics of the underlying pavement soils. Table 4.4 describes the strength classification of soils as per the DCP test index.

TABLE 4.4 Strength Characteristics of Soils as per DCP Test Index

Number of blows (inches/blow)	Strength characteristics of soils
1	Poor
0.5	Good
0.1	Excellent

The basis of the extended CBR method is the U.S. Army Corps of Engineers equation. The pavement thickness is obtained as function of load (or equivalent single-wheel load for multiple wheels), the contact area or pressure, the CBR of the subgrade, and the number of load coverage (Uzan et al. 1996). United States Army Corporation of Engineers (USACE) correlates DCP test index to CBR value of stabilized soil, through the following correlation developed.

$$CBR = \frac{292}{PR^{1.12}}$$

Table 4.5 gives correlation between DCP test index and CBR values for all the soil types except CL and CH.

TABLE 4.5 Correlation of DCP Index to CBR, Soil Types Other Than CH or CL (http://www.afcesa.af.mil/userdocuments/publications/ETL/ETL%2002-16.pdf)

DCP Index mm/blow	In/Blow	CBR	DCP Index mm/blow	In/Blow	CBR
< 3	0.1	100	6	0.27	34
	0.11	92		0.28	32
	0.12	84		0.29	31
3		80		0.3	30
	0.13	76		0.35	25
	0.14	70		0.4	22
	0.15	65	10-11		20
	0.16	61		0.45	19
4		60	12		18
	0.17	57		0.5	17
	0.18	53	13		16
5	0.19	50	14	0.55	15
	0.2	47		0.6	14
	0.21	45		0.65	13
	0.22	43		0.7	12
	0.23	41	18-19		11
6		40	20-21	0.8	10
	0.24	39	22-23	0.9	9
	0.25	37	24-26	1	8
	0.26	35			

From the criteria described in table 4.4, it is evident that for rubblization, subbase soils should have DCP test index of 0.5 or less. Utility curve has also been developed for the strength of subbase soils developed on the basis of DCP test index as shown in Table 4.5. The nature of utility curve is convex upwards (Ledbetter et al. 1979), which is mainly due to the inverse relationship between Utility and the DCP test index.

Subgrade Drainage

Drainage is one of the most important considerations in designing and constructing the pavements. As the water content of bases and subbases increases, there is a reduction in supporting power and an increase in the rate of loss of serviceability of pavements (Cedergreen et al. 1974). As rubblization technique involves breaking of concrete slab, it is required to have adequate pavement support conditions. Drainage for rubblized pavement is important (Cervarich et al. 2001) in order to prevent distresses resulting from insufficient drainage and poor support conditions and drainage conditions. In areas of high water table appropriate drainage system should be established at least 3 to 4 weeks prior to rubblization. The purpose of subsurface drains for highway pavements is to ensure maximum practical protection from free water (Cedergreen et al. 1974). Hence drainage system should also be capable of removing water from rubblized concrete layer, base layer and subgrade during rubblization. For rubblization, sufficient provision of road ditches along adequate permeability of base soils is very essential to drain of the excess water from the pavement layers. Pavements should also have adequate cross slopes for rubblization to avoid driving problems and formation of sheet flow on the pavement surface. Finally all the rubblized pavement joints should be properly sealed so that no incompressible material or water infiltrates the pavement layers from the surface of the joints. Hence all of the above four parameters have been considered to develop drainage criteria that would govern the selection of rubblization technique in order to prevent the occurrence of distress due to poor drainage conditions and inadequate support conditions.

<u>Cross Slope</u>

Adequate cross slope of the pavement surface is very essential for proper drainage in pavement. Rubblization requires adequate pavement drainage conditions for its proper functioning. Hence adequate provision of cross slopes is very essential to prevent the formation of sheet flow on the rubblized pavement surfaces to aid proper drainage and avoid occurrence of driving hazards. The ID Model developed by Liu and Lytton has been used for the cross slope analysis. Originally developed by Casagrande and Shannon, Infiltration and Drainage (ID) Model (Lytton et al. 1990) is a program that does three main evaluations: Drainage analyses, Pavement Evaluation and System analysis of rainfall and Infiltration and Drainage. However, Liu and Lytton (Lytton et al. 1990) modified the model for computing the degree of drained area versus time for saturated granular base course with lateral drainage overlying permeable or impermeable subgrade.

The ID Model modified by Liu and Lytton (Lytton et al. 1990) shows that for soils in base course with 85% saturation, the acceptable time for drainage is approximately 10 hours. Using this criteria relationship between slope factor *S* and the degree of drainage was studied using Casagrande-Shannon Model (Lytton et al. 1990). The analysis suggests a slope factor of 0.1 for rubblization of concrete pavements. A utility curve (Ledbetter et al. 1979) has also been developed for slope factor based upon the above criteria.

Permeability

Soil permeability is the ease with which liquid, gases or plant roots pass through the given layer of soil. Thus permeability is a soil property which expresses or describes how water flows through soils (Holtz and Kovacs 1981). The amount of water that flows in soil is related to the amount of excess pressure added to the bottom and soil permeability (Lambe et al. 1979). Rubblization requires adequate drainage of rubblized concrete layer, base layer and subgrade during construction. The rate at which water drains from the base course depends upon the permeability of the soil and the hydraulic gradient. Hence soil permeability is one of the important considerations for rubblization technique. As described above, the ID Model modified by Liu and Lytton (Lytton et al. 1990) showed that soils in base course with 85% saturation have approximately 10 hours as the acceptable time for drainage. This criterion was used to establish relationship between soil permeability *K* and the degree of drainage was studied using results from ID model modified by Liu and Lytton (Lytton et al. 1990). Soil permeability *K* is defined as the ratio of permeability of subgrade to the base course. The analysis showed that as the ratio of subgrade to base permeability increases, the drainage characteristics of the pavement gets better. The K value of 0.1 or less is suggested for the rubblized pavements. Table 4.6 gives permeability values of soils for all the three major groups. The soil classification in Table 4.6 is based on Unified Soil Classification System, (USCS) which originally developed by Professor A. Casagrande was modified by US Army Corps of Engineers (USACE) to make the system applicable to dams, foundations and constructions (Holtz and Kovacs 1981).

A utility curve (Ledbetter et al. 1979) has been developed for the soil permeability K with respect to degree of drainage on basis of the relationship shown in Liu and Lytton Model (Lytton et al. 1990).

Joint Seal Damage

Joint seal damage is any condition that enables incompressible materials or water to infiltrate the joint from the surface, et al LTPP Distress Identification Manual. The damage mainly includes disintegration, removal, pull out, hardening or debonding of the joint material from the adjoining slab edge. Due to this, water easily enters the pavement surface thereby being available to erode the subbase layers. Table 4.7 gives LTPP Distress Identification Manual description of various severity types for joint seal damage.

TABLE 4.6 Characteristics of Soils Pertaining to Embankments and Foundations (www.adtdl.army.mil/cgi-bin/atdl.dll/fm/5-410/Ch5.htm)

Major Divisions (1)	(2)	Letter (3)	Symbols: Hatching (4)	Symbols: Color (5)	Name (6)	Value for Embankments (7)	Permeability cm per sec (8)
Coarse-Grained Soils	Gravel and Gravelly Soils	GW		Red	Well-graded gravels or gravel-sand mixtures, little or no fines	Very stable, pervious shells of dikes and dams	$k > 10^{-2}$
		GP		Red	Poorly graded gravels or gravel-sand mixtures, little or no fines	Reasonably stable, pervious shells of dikes and dams	$k > 10^{-2}$
		GM		Yellow	Silty gravels, gravel-sand-silt mixtures	Reasonably stable, not particularly suited to shells, but may be used for impervious cores or blankets	$k = 10^{-3}$ to 10^{-6}
		GC		Yellow	Clayey gravels, gravel-sand-clay mixtures	Fairly stable, may be used for impervious core	$k = 10^{-6}$ to 10^{-8}
	Sand and Sandy Soils	SW		Red	Well-graded sands or gravelly sands, little or no fines	Very stable, pervious sections, slope protection required	$k > 10^{-3}$
		SP		Red	Poorly graded sands or gravelly sands, little or no fines	Reasonably stable, may be used in dike section with flat slopes	$k > 10^{-3}$
		SM		Yellow	Silty sands, sand-silt mixtures	Fairly stable, not particularly suited to shells, but may be used for impervious cores or dikes	$k = 10^{-3}$ to 10^{-6}
		SC		Yellow	Clayey sands, sand-silt mixtures	Fairly stable, use for impervious core or flood-control structures	$k = 10^{-6}$ to 10^{-8}
Fine-Grained Soils	Silts and Clays LL < 50	ML		Green	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Poor stability, may be used for embankments with proper control	$k = 10^{-3}$ to 10^{-6}
		CL		Green	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Stable, impervious cores and blankets	$k = 10^{-6}$ to 10^{-8}
		OL		Green	Organic silts and organic silt-clays of low plasticity	Not suitable for embankments	$k = 10^{-4}$ to 10^{-6}
	Silts and Clays LL $\geq$ 50	MH		Blue	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Poor stability, core of hydraulic-fill dam, not desirable in rolled-fill construction	$k = 10^{-4}$ to 10^{-6}
		CH		Blue	Inorganic clays of high plasticity, fat clays	Fair stability with flat slopes, thin cores, blankets and dike sections	$k = 10^{-6}$ to 10^{-8}
		OH		Blue	Organic clays of medium to high plasticity, organic silts	Not suitable for embankments	$k = 10^{-6}$ to 10^{-8}
Highly Organic Soils		Pt		Orange	Peat and other highly organic soils	Not used for construction	

NOTES: 1. Values in columns 7 and 11 are for guidance only. Design should be based on actual test results.

2. The equipment listed in column 9 will usually produce the desired densities with a reasonable number of passes when moisture conditions and thickness of lift are properly controlled.

3. The range of dry unit weights listed in column 10 are for compacted soil at OMC when using the Standard Proctor Test (ASTM 1557-91).

TABLE 4.7 Type of Severity Level for Joint Seal Damage (LTPP Distress Identification Manual, June 2003)

Type of Severity	Joint Seal Damage
Low	Less than 10%
Medium	Between 10 % to 50%
High	Over 50%

From the above criteria, it is evident that joint seal damage of 40% or less is essential for the proper drainage characteristics for the rubblized pavements. A Utility curve has also developed for joint seal damage on the basis of above criteria. The nature of this utility curve is convex upward (Ledbetter et al. 1979), which is mainly due to the inverse relationship between Utility and the joint seal damage (in percentage).

Flow Line of Roadside Drainage

The main function of roadside ditch is to efficiently drain the excess water from the pavement layers without damaging ditch system, adjacent roadway or the abutting property and hence provide maximum practical protection to pavement layers from free water. As it is not possible to determine all spots from where the water will seep in, the roadside ditches must be provided in parallel along the full width of the entire traveled pavements that would be subjected to heavy traffic load. The vertical distance of flow line below the road surface is very important for the proper drainage of water from the base or subbase layers of the pavement. Table 4.8 gives the type of drainage condition with respect to the average vertical depth of flow line below the road surface.

TABLE 4.8. Type of Drainage Conditions as per Depth of Flow Line Road Surface

Type of Drainage	Vertical Depth of Flow Line below Road Surface
Low	Less than 2 feet
Medium	Between 2 feet to 4 feet
High	5 feet or more

From the above criteria, it is evident that the vertical depth of flow line below the road surface should be five feet or more for the proper drainage characteristics of rubblized pavements. A utility curve has also been developed for vertical depth of flow line below road surface on the basis of above criteria. The nature of the utility curve is concave upward (Ledbetter et al. 1979), which is mainly due to the direct relationship between Utility and the vertical depth (in feet) of flow line below the road surface.

SUITABILITY CONSIDERATIONS

After the rehabilitation strategy passes the feasibility and acceptability criteria, they are further screened for suitability considerations. Suitability criteria are based on:

- Overall pavement condition (OC) rating
- Life extension rating (user-defined).
- First cost rating (user defined).
- Time of construction rating (user defined).

The overall condition rating, as described in Figure 4.2, is based upon a weighted sum of the structural and functional rating as noted at the bottom of Table 4.3. The remaining ratings are user defined depending upon, in the user's opinion, how well the treatment combination satisfies the project requirements for life extension, cost, and time of construction. As previously noted, the time of construction rating will depend upon the traffic level.

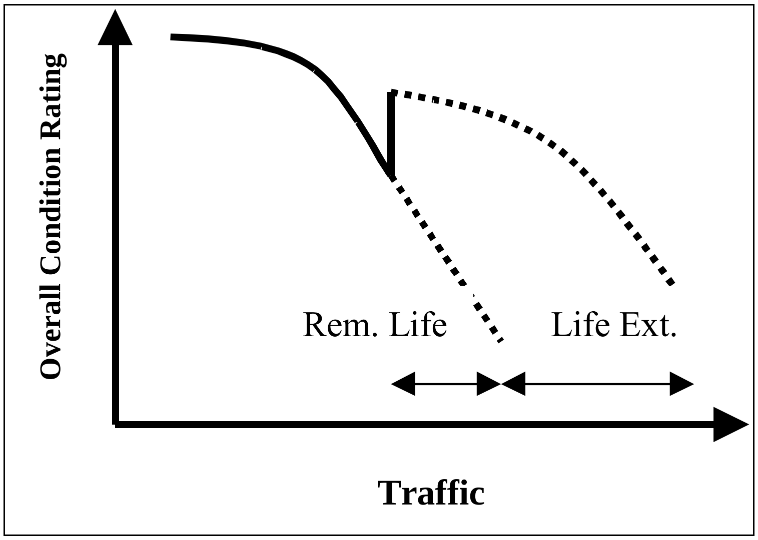

FIG. 4.2. Condition Rating Projections
(Repair and Rehabilitation of Concrete Pavement, FHWA 2001)

Once the rating (U_i) for each of these is determined, weighted values (w_i) are assigned leading to the determination of a combined rating. Finally, a combined value is generated as an integral part of the selection process for each alternate MRR technique by assigning a user defined weighting factor to the rating of each individual component allowing for all rating values to be combined into a single parameter as follows:

$$U(a) = \sum_{i=1}^{n} w_i U_i(a) \tag{4.2}$$

where:

$U_i(a)$	=	the average rating of the *i*th attribute component
w_i	=	the normalized weight of the *i*th attribute component
$U(a)$	=	the overall expected rating
a	=	the number of attribute components

If the final rating value (U(a)) is greater than a pre-selected value (say 50% of the scale value), then the strategy could be considered to be suitable and worthy of further consideration relative to LCC and other considerations. The user would of course define this pre-selected value. It is also pointed out that the amount that each distress type is weighted may depend upon the type of strategy to be applied. To facilitate the selection of weighting values, tables in appendix D are provided to suggest weights for each attribute component as a function of the strategy type.

As previously indicated, if the strategy type is routine maintenance, then apparently the only relevant factor is the cost of the rehabilitation work and the amount of coverage that can be achieved with it. For this reason, the selection of treatment type is cost driven. Most SHAs have standard procedures to select pavement treatments. These procedures consider traffic and other construction issues at very summarized level (Anderson et al. 2002). The treatment type to be selected for routine maintenance is often user defined according to SHA policies and practices. It is also pointed out that the funds for routine maintenance may also be fixed amounts appropriated for a set period of time and established for more than just MRR project work. Therefore, it may be possible to establish a utility rating curve for the use of such funds based upon the ratio of the project cost to the amount available. On this basis, the propensity to execute maintenance work with a high ratio would be greater later in the fiscal year rather than earlier. Consequently, smaller and lower cost maintenance repairs are executed earlier in the year rather than later since the more costly projects are delayed until there is evidence that project funds are available to cover other 'must do' maintenance work.

As a final note relative to the acceptability criteria, the determination of both remaining life and life extension should be based upon the structural characteristics of the existing pavement system and the results of the pavement evaluation process. These determinations are made using performance models or other similar tools, but pavement evaluation data is typically analyzed with respect to projected traffic levels in order to estimate the expected remaining life in the pavement. Additionally, life extension also needs to be estimated based upon the improvements afforded by the selected pavement treatment. It is evident that pavement life is dependant upon the stiffness of the pavement system (as depicted by the radius of relative stiffness value) and the degree that CPR or similar repairs extent pavement life depends upon the extent the stiffness of the pavement system is restored. This effect is critical when considering the impact, for instance, of full depth repairs made with Portland cement concrete versus asphalt concrete on the projection of service life. This determination is important from the standpoint that the projection of service life is an essential part of the life-cycle cost analysis which is depicted by the different types of pavement condition indices noted in table 4.3. Based upon the projection of the various distress types (depending upon the type of pavement involved), the pavement condition ratings can be projected relative to traffic using performance-rating relations similar to those shown in figure 4.2

Pavement Rehabilitation Strategy Selection ("Pavement" 1993) describes guidelines for selecting cost-effective rehabilitation strategy and provides systematic decision making process (DMP) based on construction. Recent version of American Concrete Pavement Association (ACPA) guide has also addressed traffic issues by developing handbook on traffic management ("Traffic" 1998). Also manual procedures for calculating work zone and road users costs (Walls and Smith 1998) have been documented in Appendix F.

CHAPTER V
CONCLUSION

SUMMARY

Due to the increasing traffic volumes and continuous aging of the pavements under varying climatic conditions, maintenance, rehabilitation and reconstruction (MRR) have become the principal concern for many state highway agencies. Previous research has identified feasibility, acceptability and suitability criteria that every rehabilitation strategy should meet. The main focus of this research is to append additional guidelines to these criteria so as to provide a comprehensive tool to state highway agencies for selecting the optimum MRR strategies.

The literature review identified and compiled information on various distresses in concrete pavements in order to comprehend the mechanism behind the formation of distresses in concrete pavement and the way they affect the pavement performance. Literature review also focused on various pavement repair treatments used for minimizing the occurrence of reflective cracking in asphalt overlays laid on the concrete pavements.

Based on the information collected, the concepts involved in the process of MRR strategy selection were appropriately defined and analyzed. Previous researches have identified feasibility, acceptability and suitability criteria that every MRR strategy should satisfy. The MRR strategies satisfying these criteria are then weighed in decision process to determine the optimum rehabilitation strategy. Research also focused on various repair treatments that would minimize the reflective cracking in the pavements and identified the lessons to be considered for the future projects.

CONCLUSION

The research has appended the decision guidelines developed for selecting strategies for Maintenance, Rehabilitation and Reconstruction (MRR) of rigid pavements subjected to high traffic volumes. The decision guidelines generally fit with the construction practices of state highway agencies and would help them in selecting the optimum rehabilitation strategy for pavement treatment.

The research first focused on addressing questions that influence the performance of pavements. The first question concerned the mechanism behind the formation of various distresses and the way they affect the pavement performance. These were identified through the literature review and the data collection process. The second question concerned evaluating the assessing the existing pavement treatments for minimizing reflective cracking in pavements.

The third question focused on developing recommendations for statewide methods for rehabilitating jointed concrete pavement so as to minimize reflective cracking. Research performed literature review so as to evaluate the existing methods used in present industry for controlling reflective cracking in pavements. Research then focused on appending the existing guidelines for selecting the optimum Maintenance, Rehabilitation and Reconstruction (MRR) strategy for pavement treatment. The decision criteria necessary for selecting the optimum rehabilitation treatment mainly included feasibility, acceptability and suitability criteria. Feasibility criteria imply minimum conditions with regard to service life, traffic and climatic conditions that every selected rehabilitation strategy should first meet. Acceptability criteria are based on the structural and functional capacity contribution provided by each pavement type. Research further added soil strength and drainage criteria to the existing acceptability criteria. Suitability criteria mainly include cost-effective, life extension and non-pavement related parameters associated with each rehabilitation alternative.

The selected pavement rehabilitation alternatives are first screened for feasibility criteria. The feasible alternatives are then screened for acceptability considerations. The technically feasible and acceptable alternatives are then screened for suitability criteria. The rehabilitation strategies satisfying all the above criteria are then weighed in a decision process to determine the optimum rehabilitation strategy.

The developed MRR strategy guidelines are fairly generic and needs to be extended to a more detailed level. This would provide state highway agencies with a more easy and comprehensive tool to help in selecting the optimum pavement rehabilitation treatment. A set of comprehensive guidelines in identifying various procedures and restraints that state highway agencies would use for selecting MRR strategies would be the future approach of this research.

RECOMMENDATIONS

The early version of Rigid Pavement Rehabilitation Design System (RPRDS) was developed by Center of Highway Research (Seeds et al. 1982). EXPEAR (Hall et al. 1989) developed in 1989 by University of Illinois performed project-level evaluation on the basis of visual survey conditions and recommended various rehabilitation techniques. But it never considered user costs and other indirect impacts of recommended rehabilitation techniques. Previous research has identified feasibility, suitability and acceptability criteria for selecting the optimum Maintenance, Rehabilitation and Reconstruction (MRR) strategies. The result of the research was a computerized program SAPER (Repair and Rehabilitation of Concrete Pavements, FHWA 2001). The current research focused on appending the additional criteria to the previously established criteria. My recommendations for future research will be to update SAPER to include the additional decision criteria developed as a part of current research.

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APPENDIX A

ANNOTATED BIBLIOGRAPHIES

Title: Nationwide Evaluation Study of Asphalt Concrete Overlays Placed on Fractured Portland Concrete Pavements

Authors/Agency/Country: Witczak-MW; Rada-GR

Source: Transportation Research Record. 1992. (1374) pp (19-26)

Report Number/Date: 0309054168

Number Of Pages: 8

Summary:

The paper presents the general approach and evaluation of a nation wide study conducted to evaluate the performance of various rehabilitation strategies for eliminating reflective crack. The research study consisted of 93 sections- 17 rubblized sections, 35 crack/seat and 41 break/seat projects. Data on field performance, deflection measurements and performance predictive equations is presented. Some of the important findings of the research included developing predictive models for fractured PCC modulus for rubblization and crack and seat technique and evolving statistical frequency distributions for effective PCC modulus (E_{PCC}) for all the rehabilitation techniques including between project and within project variability. The research study concluded rubblization and AC overlay as the best rehabilitation technique for increasing the performance life of the pavements in all the PCC pavement types. The research study also identified Crack and Seat technique and AC overlay to be an effective rehabilitation technique for JPC pavements.

Title: Structural Adequacy Of Rubblized Portland Cement Concrete Pavement

Authors/Agency/Country: Galal-KA; Coree-BJ; White-TD

Source: Transportation Research Record. 1999. (1684)

Report Number/Date: 0309071097

Number of Pages: 7

Summary

This report reviews a rubblization project conducted by INDOT to evaluate the structural adequacy of the rubblized pavement. Rubblization project was conducted on a portion of US-41 section, Benton County. The rehabilitation task was mainly divided into two sub contracts: The northern subcontract and the southern subcontract. The results of the project showed AASHTO layer coefficient of 0.25. This layer coefficient was two standard deviations less than that reported by PCS/Law. As INDOT used a layer coefficient of 0.2, this layer coefficient was set within the two standard deviation limits of the INDOT layer coefficient. The final layer coefficient of 0.22 was a conservative and reasonable value and was recommended for rubblized pavements. It was recommended that INDOT should conduct further similar studies if it wanted to use rubblization technique.

Title: Rubblization Of Concrete Pavements

Authors/Agency/Country: Ksaibati-K; Milewy-W; Armaghani-J

Source: Transportation Research Record. 199. (1684)

Report Number/Date: 0309071097

Number Of Pages: 7

Summary

This report presents a summary of the results of nation wide survey performed by Florida DOT to collect the information about the practices performed by other state DOT’s with regard to rubblization and also determine the performance of rubblized sections in various states. The survey results indicated that most of the states had relatively less

number of rubblized sections, with the exception of three states that had more than ten sections each. Also wide variation was seen in the practice of construction techniques, overlay thicknesses and field performance among the various states. The survey results indicated that most of the states were satisfied with rubblization as a rehabilitation technique to eliminate the reflective cracking in all the types of concrete pavements.

Title: Hot-Mix Asphalt Overlay Design Concepts for Rubblized Portland Cement Concrete Pavements.

Authors/Agency/Country: Thompson – MR

Source: Transportation Research Record. 1999. (1684)

Report Number/Date: 0309071097

Number Of Pages: 9

Summary

This report presents the analysis and design concepts of Illinois State Department of Transportation's (IDOT) for HMA overlay thickness on rubblized concrete pavements. IDOT had excellent performance from its first rubblization project on I-57 near Pesotum. The current concern is to determine the appropriate HMA OL thickness for range of project variables. IDOT uses M-E flexible pavement design concepts for rubblized PCCP's. HMA fatigue considerations is the principal criterion for overlay thickness requirements for rubblized PCCP's. Fatigue life is predicted on the basis of estimated HMA strain and HMA design fatigue algorithm. The M-E design approach used by IDOT is based on the concept of "Design Time" AC temperature modulus. The analysis of various data showed that AUPP FWD deflection basin parameter was the best parameter for estimating HMA OL flexural strain. The specification of rubblized material (coarse or fine) governs the behavior of rubblized PCCP. HMA fatigue properties controls the relative life calculations for HMA OL thickness design.

Title: How WisDOT Uses Rubblization

Source: Better Roads. 2000/02.

Report Number/Date: 70/02

Number Of Pages: 2

Summary

This report describes the Wisconsin State Department of Transportation's (WisDOT) unique specification for rubblization. Reflective cracking is the major concern for the concrete pavements rehabilitated with asphalt overlays. WisDOT specification requires that the top layer of the rubblized concrete must have stones approximately two inches in diameter, middle layer of the rubblized concrete must have stones approximately three inches in diameter and bottom four-inch layer must have stones not greater than twelve-inch diameter. In the cases where poor subgrade conditions were encountered, the speed of rubblizing machine was increased and less effort was pounded on pavement. This resulted in larger concrete pieces of 6-inch to 8-inch diameter but the quality of the obtained crushed stone was consistent. The machine was also sped on manholes. The speed of machines was varied on the quality of concrete and the diameter of aggregate. After rubblization, concrete was seated with initial two or three passes of vibratory grid roller. Rubber tire roller was then used to remove uneven areas due to vibratory grid roller. Finally steel wheel roller was used to eliminate the unevenness of the pavement. WisDOT designs pavements considering truck traffic (single axle, 18 kips, 1000lb. Loading). The long-term performance information provided by this study will determine the overall life cycle cost of rubblization with asphalt overlay.

Title: Positive Experience with Rubblization of Rigid

Authors/Agency/Country: Alan Rommel

Source: Wisconsin Asphalt Pavement Association (WAPA), 1999.

Summary

This report documents the construction details of rubblization on USH 12 project, Waukesha district, Wisconsin. The 8.6-mile stretch of 4 lane divided highway had severe joint deteriorations for longitudinal and transverse joint deteriorations. Breaking was seen in the joints and the shattering of concrete surface was causing safety concerns of the road. Crack and Seat with an overlay was the first chosen rehabilitation alternative. But problems arose as cracking and seating of the pavement proceeded. After brief discussion rubblization was chosen as the final alternative for pavement rehabilitation. Several meetings were held with experienced contractors, Wisconsin Asphalt Pavement Association (W.A.P.A) and other DOT offices to collect all the inputs and information regarding rubblization. After this a design team was formed that formulated a construction plan including under drain, rubblization and an asphalt overlay which would solve the pavement concerns of USH 12 and minimize the traffic delay. B.R. Amon & Sons initiated the construction on July 1998. Antigo Construction performed the rubblizing operation with a multi head breaker (M.H.B). The rubblizing operation was carried out in the specified time limit and as per the standard specifications. The project was completed in September 1998 at a cost of $ 7.5 million. Profilograph reading showed a ride quality index of 2 inch per mile indicating a very good rideability of the pavement section. Also no reflective cracking had been seen in the pavement section. Hence the Wisconsin DOT had a successful experience with rubblization in terms of delivery cost, time for design and construction and overall quality of the project.

Title: HMA Overlay Construction With One Pass/Lane Width PCCP Rubblization

Authors/Agency/Country: Thompson-M; VanMatre-F; Lippert-D; Jenkins-P

Source: Conference Title: Asphalt Paving Technology

Report Number/Date: 00752125

Number Of Pages: 19

Summary

The report documents the construction details of the rubblization project at the northbound lanes of I-57 Effingham County, Illinois DOT. Originally constructed in 1970, the 8-inch CRCP laid over 4 inch bituminous aggregate mixture (BAM) subbase was experiencing severe D-cracking distress. Pre-construction evaluation of the pavement done in 1995 indicated that the pavement had adequate subgrade strength to support rubblization and HMA overlay but was vulnerable to rapid D-cracking deterioration. Two rehabilitation alternatives were considered: 1) Extensive PCCP patching and HMA OL and 2) PCCP rubblization and a thicker HMA OL. Rubblization and HMA OL paving operation were completed in six weeks; start to finish on the 3.2-mile long north section. Multi head breaker (M.H.B) was used for rubblizing. A vibratory roller fitted with Elliot Z-pattern grid on the roller surface followed it. Final compaction of the rubblized surface was done by one pass of rubber-tire roller and double drum vibratory roller. Milling, rubblizing, compaction and paving operation were performed in a tight sequential and continuous operation. Post construction evaluation was done by collecting the relevant FWD data and back calculating subgrade moduli on various sections. The results showed uniform rubblized sections. The deflection COV's (coefficient of variation) were low compared to the 20 percent COV value used in IDOT's full-depth HMA thickness design. The profile data obtained from IDOT's Video Inspection Vehicle on November 22, 1996, showed average IRI for the driving lane as 58 inches/mile. The average rut depth was 1.3 mm. Hence the pavement had a smooth riding surface and good rutting resistance.

Title: Before You Fix It, Break It

Authors/Agency/Country: Jill Dunlay

Source: The Asphalt Contractor, 2001

Summary

This report describes the rubblization project performed by Indiana Department of Transportation (INDOT0 on State Route 37 (S.R. 37). The 7 mile long pavement was a 7-inch thick CRCP section. Faulting was seen in the centerline joint and transverse construction joints and transverse cracking had developed in the pavement sections every 3 to 4 feet. Rubblization and overlay was chosen as the rehabilitation alternative and the contract was given to Reith-Riley Construction Co. Inc., Goshen Ind., They subcontracted rubblization task to Antigo Construction, Antigo, Wis. Prior to rubblizing

operation, new turn lanes were constructed in northbound lanes to accommodate the two-way traffic and the traffic in southbound lanes was switched into northbound lanes. Also subsurface drainage was laid prior to rubblization. On September 26 Antigo Construction started rubblizing operation with two Multi head breakers. (MHBs). The vibratory steel drum compactor from Ingersoll Rand with Z grid attachment closely followed the MHBs. Rubblizing and paving operations were closely coordinated in order to fulfill the INDOT's specifications that required rubblized pavement to be covered within 48 hours after being broken. The southbound lanes were completed in 5 days. After this the traffic flow from northbound was switched to southbound lanes. On November 4 Antigo Construction started the rubblizing operation as per the. The rubblizing and paving operations were coordinated. On areas of softer subgrade, the Antigo Construction crew increased the spacing between hammers and decreased the height of hammer drop to bridge the soft areas. The entire work was completed in approximately four days as per INDOT's specifications. The long-term performance of this project will determine the effectiveness of the rubblization and HMA overlay technique.

APPENDIX B

POSSIBLE MRR STRATEGY SELECTION FLOWCHARTS

Overview

A procedure for identifying feasible MRR strategies are given in this appendix, based on engineering criteria. The candidate MRR strategies identified using the flowcharts given in this appendix can be screened for agency and project constraints and evaluated for cost effectiveness to identify the optimum (or preferred) MRR strategy.

- Type of distress—material related, foundation movement, functional, or structural.
- Design life—short term, intermediate term, or long term.
- Traffic—average daily truck traffic (ADTT).
- Climate—LTPP climatic region.
- Pavement design—presence of edge drains.
- Pavement condition.
    - Load transfer efficiency (LTE, for JPCP and JRCP).
    - Presence of voids and the condition of the drainage system (if applicable).
    - Rate of slab cracking (JPCP) or crack deterioration (JRCP and CRCP).

Procedure for Identifying Feasible MRR Strategy

The procedure for identifying feasible MRR strategies is straightforward:

- Determine problem type:
- If materials-related distresses are present, use table D.1 to rate the severity of the problem.

TABLE B.1. Material -Related Distress (MRD) Rating
(Repair and Rehabilitation of Concrete Pavement, FHWA 2001)

		MRD Severity		
		Low	Medium	High
Extent over length of project	Less than 1/3	**1**	**1**	**2**
	Between 1/3 and 2/3	**1**	**2**	**3**
	Greater than 2/3	**2**	**3**	**3**

TABLE B.2. Trigger and Limit Values for Smoothness (ACPA 1997)

Measure	Trigger / limit values		
	Low traffic ADT < 3,000	**Medium traffic** ADT 3,000 to 10,000	**High traffic** ADT > 10,000
IRI, m/km (in/mi)	1.4 / 3.5 (89 / 222)	1.2 / 3.0 (76 / 190)	1.0 / 2.5 (63 / 158)
PSR	3.4 / 2.0	3.6 / 2.5	3.8 / 3.0
California Profilograph	18 / 100	15 / 80	12 / 60

- The criteria for functional deficiency should be established by each agency. For smoothness, the guidelines provided by the American Concrete Pavement Association are given in table B.2. The ACPA guidelines establish different trigger and limit values of smoothness for different traffic levels. The trigger value is the value at which corrective action should be considered; the limit value is the approximate practical limit for CPR.

- Enter the flowcharts with the project information and traverse until an end node is reached. Each end node in the flowchart provides a range of MRR options, listed in the increasing order of level of rehabilitation.

- Repeat step 2 until all feasible design lives have been evaluated. This step is important to ensure that all feasible MRR options are included in the evaluation.

- The feasible MRR strategies identified using the above procedures can be screened for agency and project constraints and evaluated for cost effectiveness to determine the optimum (preferred) MRR strategy using the procedures described in appendix D.

Testing and Evaluation Procedures

Table B.3 lists the recommended procedures for testing and evaluation to obtain the pavement condition data needed to evaluate MRR needs.

TABLE B.3. Listing of Testing and Evaluation Procedures (ACPA, 1997)

Protocol	Description	Applicable Standards	References
TP1	Distress survey	LTPP Distress ID Manual	SHRP 1993; Smith et al. 2001
TP16	Drainage survey		Smith et al. 2001
TP2	Deflection testing	ASTM D 4694; ASTM D 4695	Smith et al. 2001
TP3,4	Roughness testing	ASTM E 1926	Smith et al. 2001; Sayers 1990
TP7	Friction testing	ASTM E 524	Smith et al. 2001; Henry 2000

MRR Treatment Description

The following provide descriptions of the treatment groups referenced in the feasible MRR strategy selection flowcharts:

JPCP Full-Depth Repair

- JPCP FDR1.
  - All shattered slabs.
  - High-severity transverse cracks.
  - High-severity corner breaks.

- JPCP FDR2.
  - All shattered slabs.
  - Medium- and high-severity transverse cracks.
  - High-severity longitudinal cracks in the wheelpath.
  - Medium- and high-severity corner breaks.

- JPCP FDR3.
  - All shattered slabs.
  - All transverse cracks*.
  - Medium- and high-severity longitudinal cracks in the wheelpath.
  - High-severity, nonwheelpath longitudinal cracks.
  - All corner breaks.

*Low-severity transverse cracks may be retrofitted with dowel or tie bars.

JRCP Full-Depth Repair

- JRCP FDR1.
  - High-severity transverse joint spalling that extends more than 1/3 the slab thickness.
  - High-severity transverse cracks.
  - High-severity corner breaks.

- JRCP FDR2.
  - High-severity transverse joint spalling that extends more than 1/3 the slab thickness.
  - Medium- and high-severity transverse cracks.
  - High-severity longitudinal cracks in the wheelpath.
  - Medium- and high-severity corner breaks.

- JRCP FDR3.
  - Medium- and high-severity transverse joint spalling that extends more than 1/3 the slab thickness.
  - Medium- and high-severity transverse cracks.
  - Medium- and high-severity longitudinal cracks in the wheelpath.
  - High-severity, nonwheelpath longitudinal cracks.
  - All corner breaks

CRCP Full-Depth Repair

- CRCP FDR1.
  - Medium- and high-severity punchouts.
  - High-severity transverse cracks.
  - High-severity deteriorated construction joints.

- CRCP FDR2.
  - All punchouts.
  - High-severity transverse cracks.
  - High-severity deteriorated construction joints.

    - High-severity longitudinal cracks in the wheelpath.

- CRCP FDR3.
    - All punchouts.
    - Medium- and high-severity transverse cracks.
    - Medium- and high-severity deteriorated construction joints.
    - Medium- and high-severity longitudinal cracks in the wheelpath.
    - High-severity, non-wheelpath longitudinal cracks.

<u>Partial-Depth Repair</u>

- JCP PDR1—High-severity partial-depth joint spalls.
- JCP PDR2—Medium- and high-severity partial-depth joint spalls.
- CRCP PDR1—High-severity partial-depth spalls.

<u>Longitudinal Crack Load Transfer Restoration</u>

- LC1—Cross-stitch or retrofit tie bar medium-severity wheelpath longitudinal cracks.
- LC2—Cross-stitch or retrofit tie bar all medium-severity longitudinal cracks (both wheelpath and nonwheelpath longitudinal cracks).
- LC3
    - Cross-stitch or retrofit tie bar low-severity wheelpath longitudinal cracks.
    - Cross-stitch or retrofit tie bar low- and medium-severity, nonwheelpath longitudinal cracks.

<u>Preoverlay Repairs for Unbonded Overlays</u>

- JCP UPR.
    - Full-depth repair
        - All shattered slabs.
        - High-severity corner breaks

- Grind faulting > 6 mm (0.25 in) or use a thick (min 25 mm [1 in] HMA) separator layer
- Stabilize unstable slabs (undersealing).

➢ CRCP UPR

- Full-depth repair.
  - All punchouts.
  - High-severity transverse cracks.
- Underseal voids.

<u>Preoverlay Repair for Bonded Overlays</u>

➢ JPCP BPR.

- Full-depth repair.
  - All shattered slabs.
  - All transverse cracks.
  - Medium and high-severity longitudinal cracks in the wheelpath.
  - High-severity nonwheelpath longitudinal cracks.
  - All corner breaks.
- Stabilize unstable slabs (undersealing).
- Cross-stitch or retrofit tie bar the following:
  - Low-severity longitudinal cracks in the wheelpath.
  - Low- and medium-severity nonwheelpath longitudinal cracks.
- Partial-depth repair high-severity partial depth joint spalls.

➢ JRCP BPR

- Full-depth repair.
  - All shattered slabs.
  - High-severity transverse cracks.
  - Medium and high-severity longitudinal cracks in the wheelpath.
  - High-severity nonwheelpath longitudinal cracks.
  - All corner breaks.
- Stabilize unstable slabs (undersealing).
- Cross-stitch or retrofit tie bar the following:
  - Low-severity longitudinal cracks in the wheelpath.
  - Low- and medium-severity nonwheelpath longitudinal cracks.

- Partial-depth repair high-severity partial depth joint spalls.

➢ CRCP BPR
- Full-depth repair.
    - All punchouts.
    - High-severity transverse cracks.
    - Medium and high-severity longitudinal cracks in the wheelpath.
- High-severity nonwheelpath longitudinal cracks.
- Underseal voids.
- Partial-depth repair high-severity partial depth joint spalls.

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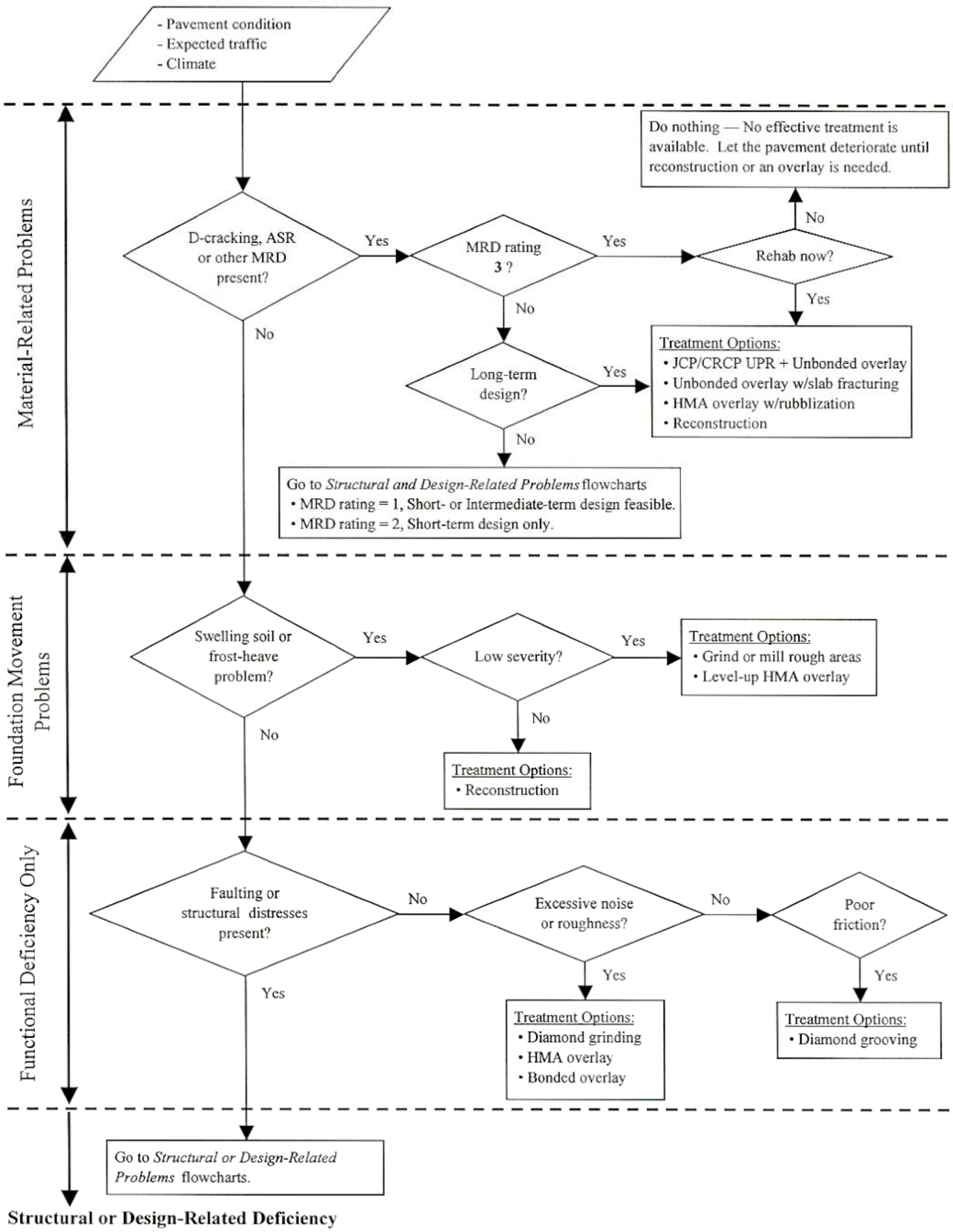
Strategic Highway Research Program (SHRP). 1993. *Distress Identification Manual for the Long-Term Pavement Performance Project*. Report No. SHRP-P338. Transportation Research Board, Washington, DC.

Possible MRR Strategy Selection Flowcharts



APPENDIX C

STRUCTURAL DISTRESS RATING CURVES

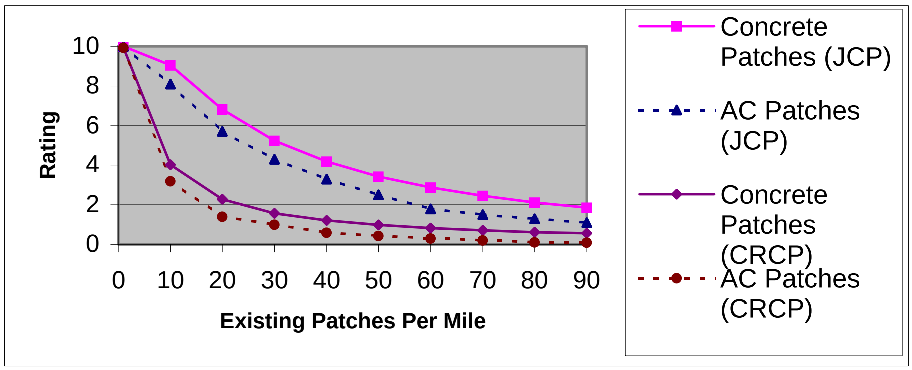


FIG. C.1. Rating Curve for Patches (Zollinger et al. 2001)

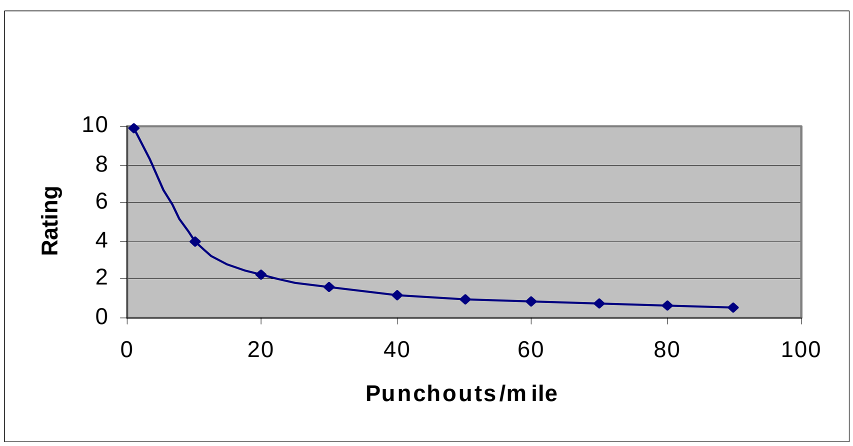


FIG. C.2. Rating Curve for Punchouts for CRCP (Zollinger et al. 2001)

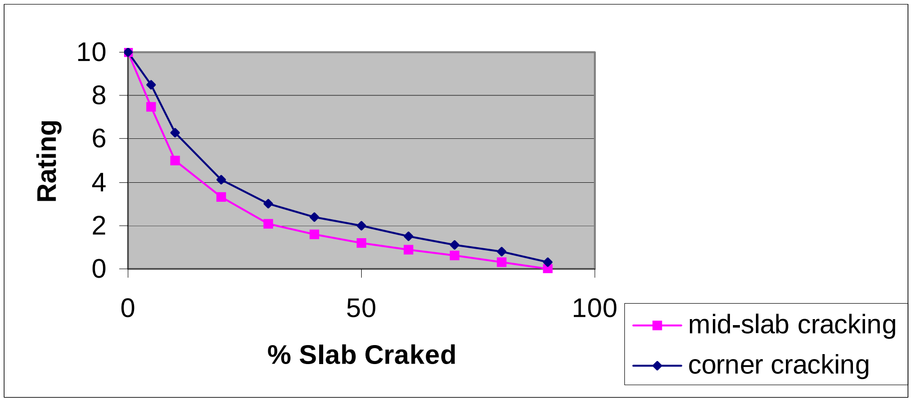


FIG. C.3. Rating Curve for Slab Cracking for JCP (Zollinger et al. 2001)

APPENDIX D

FUNCTIONAL DISTRESS RATING CURVES

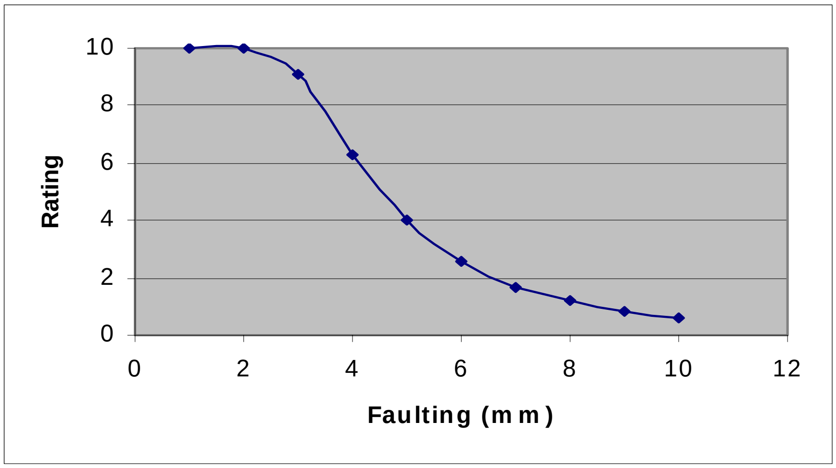


FIG. D.1. Rating Curve for Faulting for JCP (Zollinger et al. 2001)

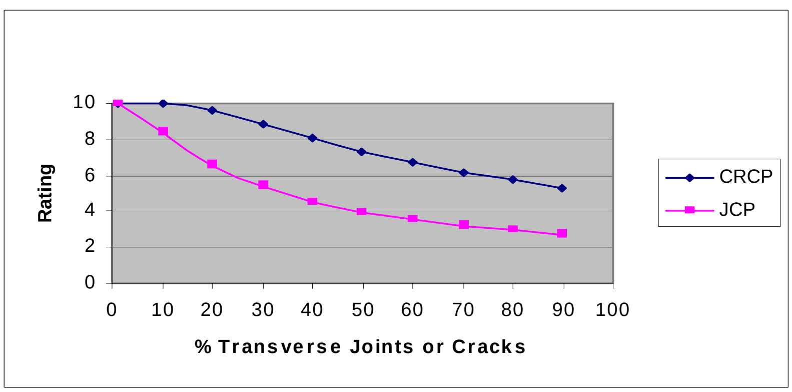


FIG. D.2 Rating Curve for Spalling (Zollinger et al. 2001)

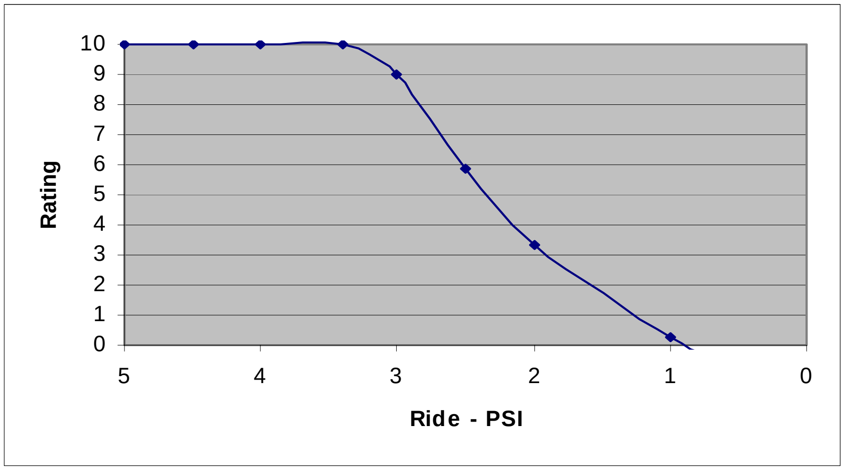


FIG. D.3. Rating Curve for Ride for JCP and CRCP (Zollinger et al. 2001)

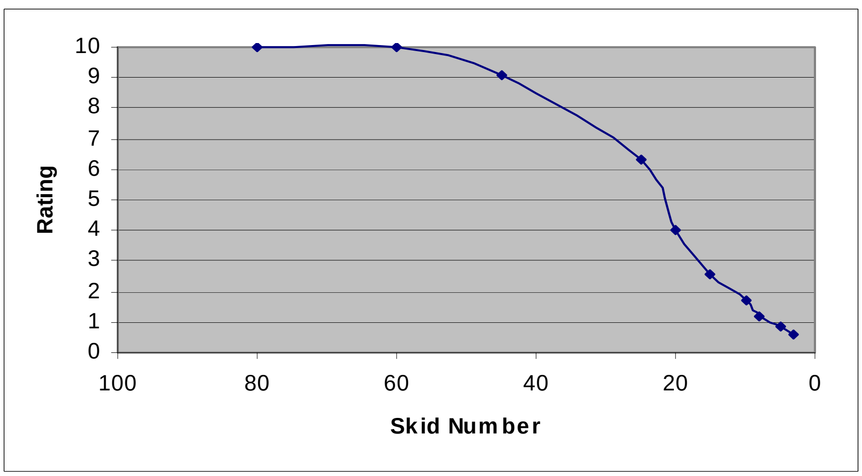


FIG. D.4. Rating Curve for Skid for JCP and CRCP (Zollinger et al. 2001)

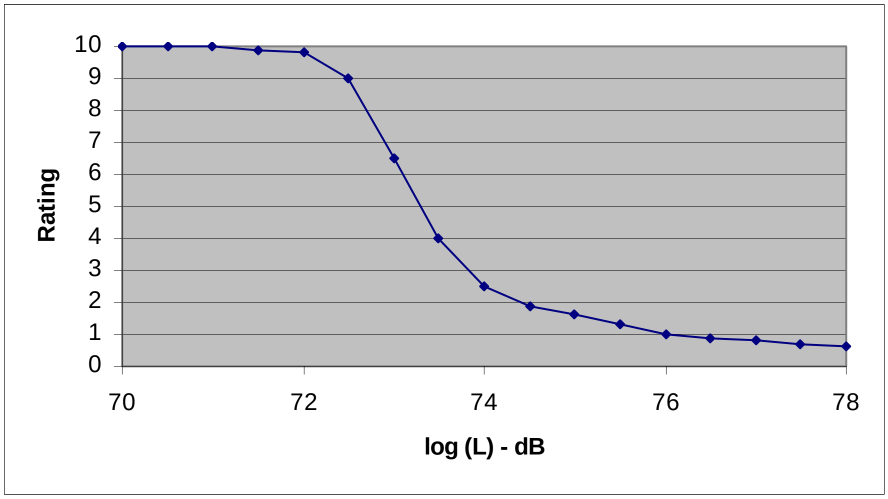


FIG. D.5. Rating Curve for Tire Noise for JCP and CRCP (Zollinger et al. 2001)

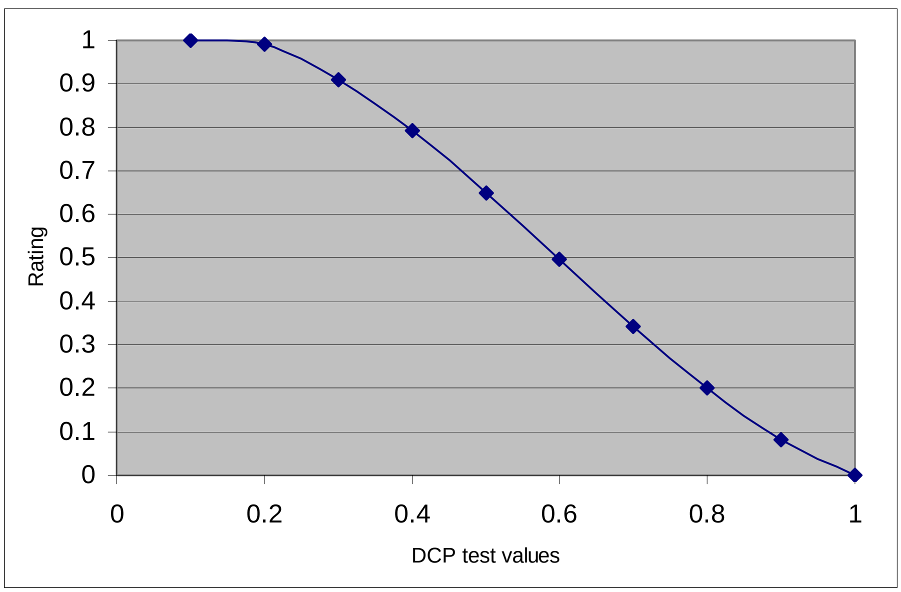


FIG. D.6. Rating Curve for DCP test values

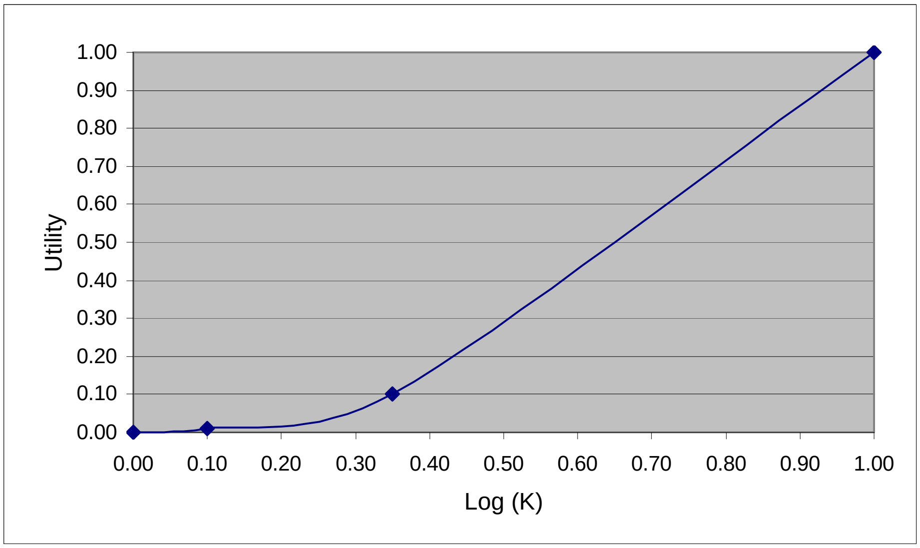


FIG. D.7. Rating Curve for Permeability

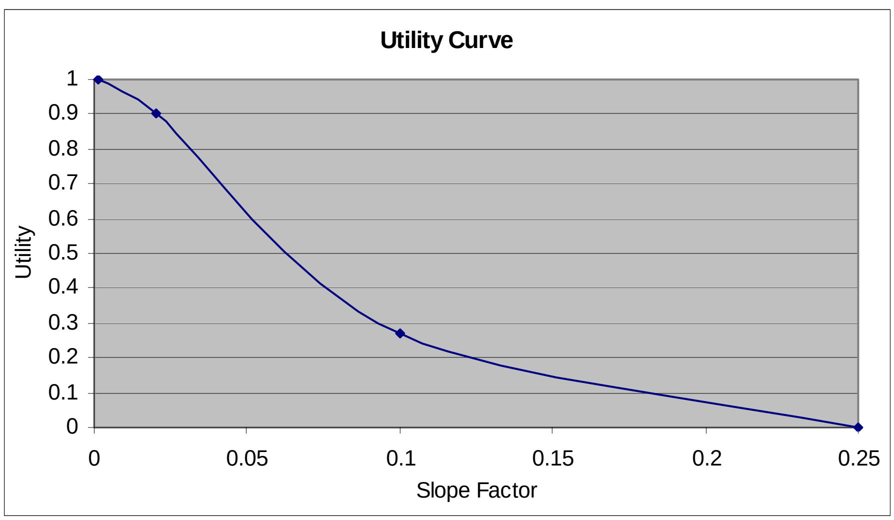


FIG. D.8. Rating Curve for Slope Factor

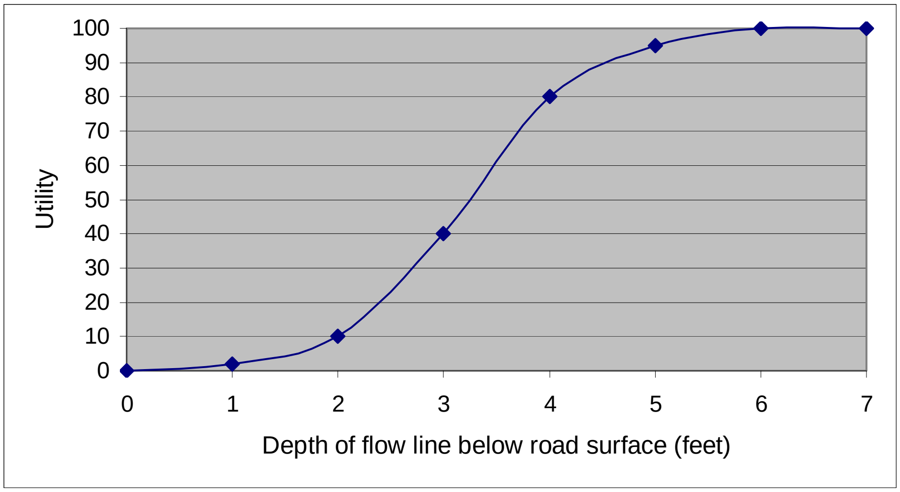


FIG. D.9. Rating Curve for Flow Line Depth

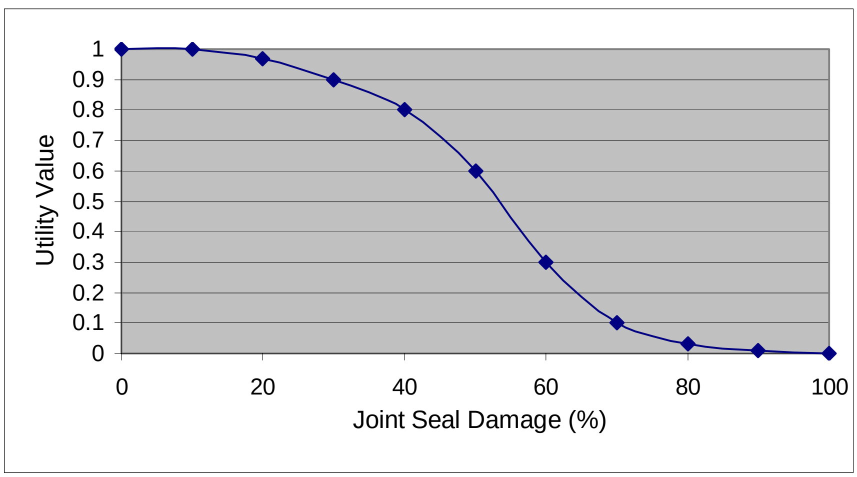


Figure D.10. Rating Curve for Joint Seal Damage

APPENDIX E

CORRIDOR IMPACT AND CONSTRUCTABILITY RATING CURVES

Corridor Impact

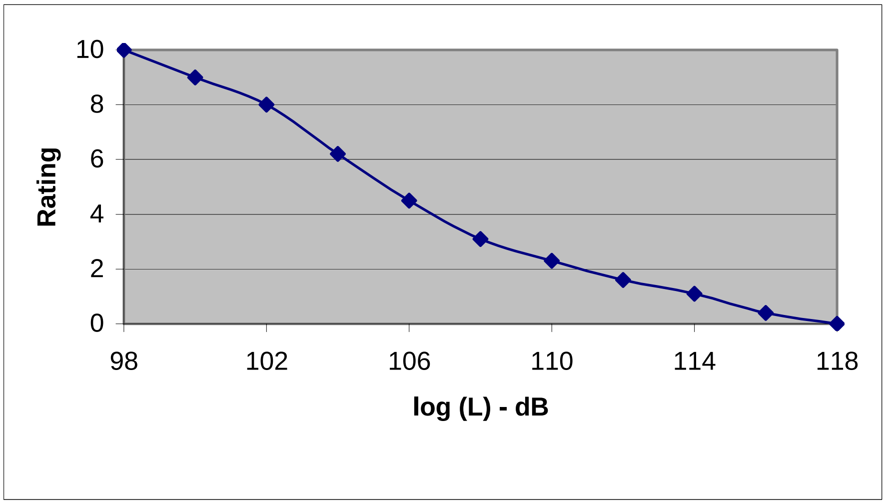


FIG. E.1. Noise (Zollinger et al. 2001)

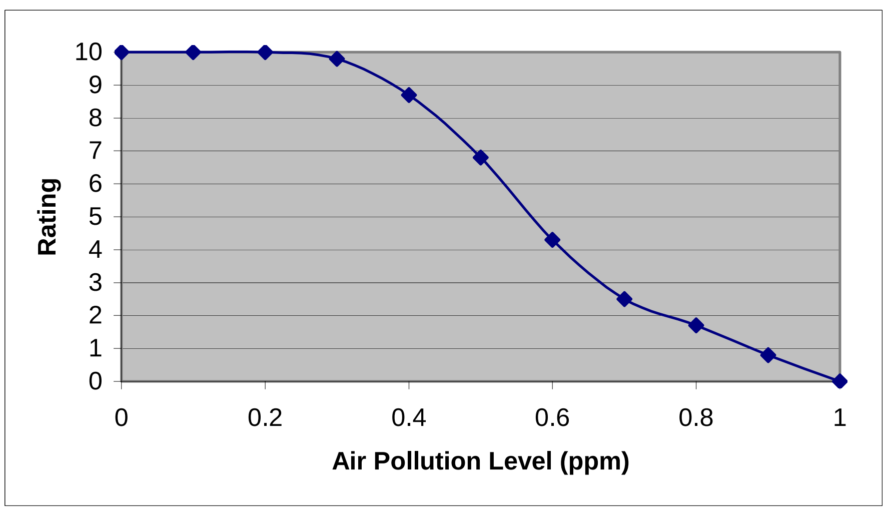


FIG. E.2. Air Pollution (Zollinger et al. 2001)

Constructability

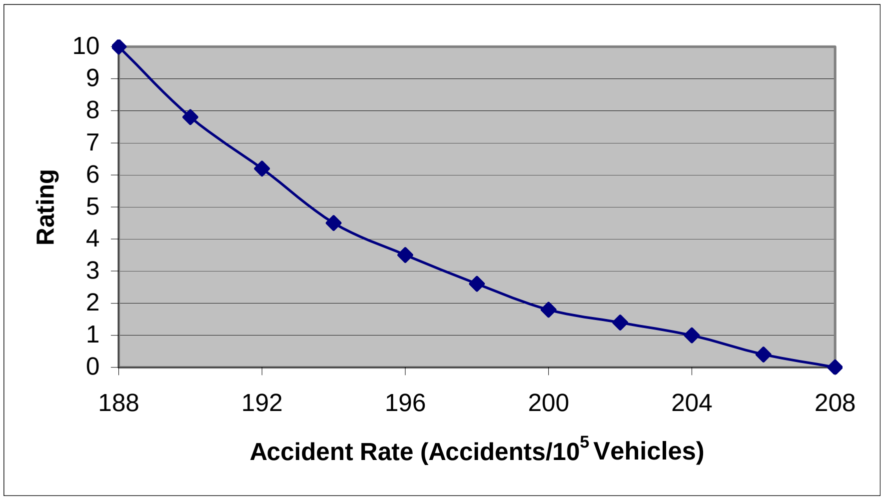


FIG. E.3. Accident Rate in terms of Property Damage (Zollinger et al. 2001)

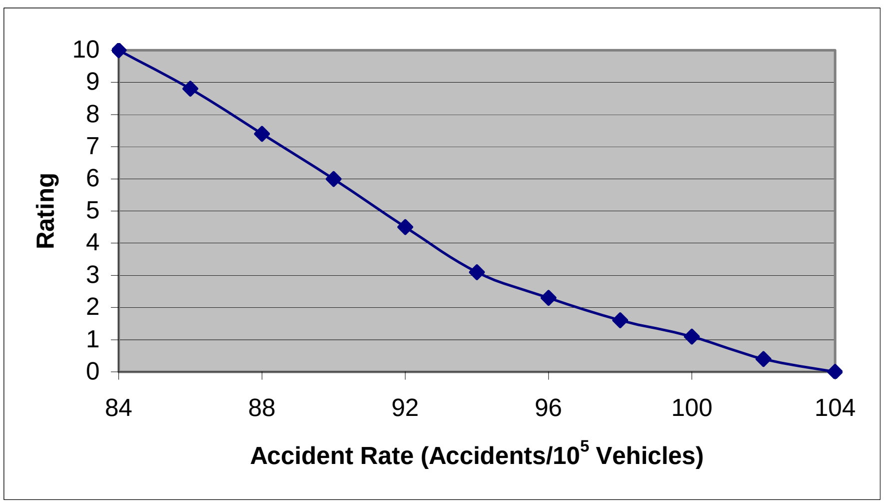


FIG. E.4. Accident Rate in terms of Physical Injury (Zollinger et al. 2001)

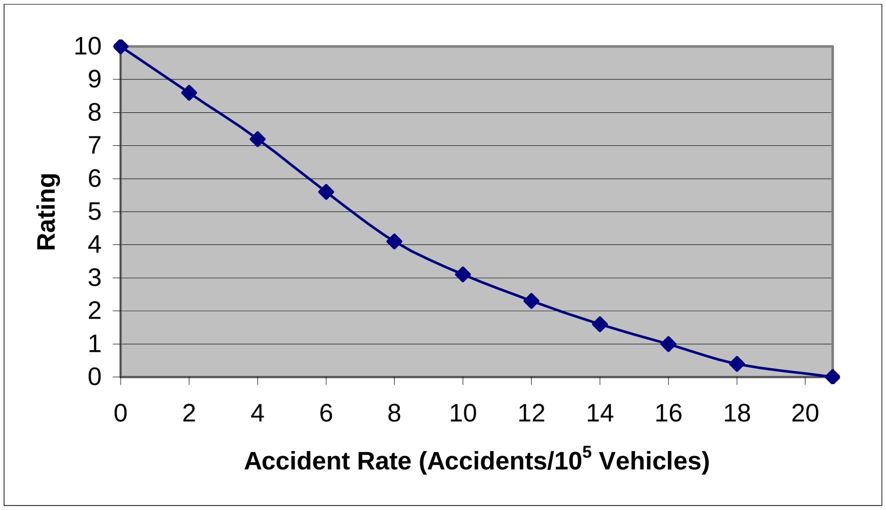


FIG. E.5. Accident Rate in terms of Fatalities (Zollinger et al. 2001)

Concrete Pavement

Scale
5 - Superior **4 - Excellent** **3 - Good** **2 - Fair** **1 - Poor** **0 - None**

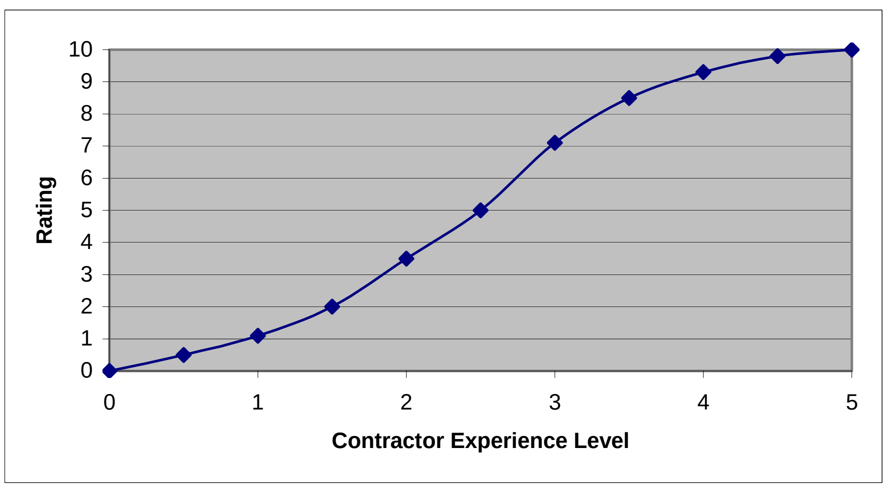


FIG. E.6. Contractor Experience for Concrete Pavement (Zollinger et al. 2001)

Scale

5 - Superior
4 - Excellent
3 - Good
2 - Fair
1 - Poor
0 - None

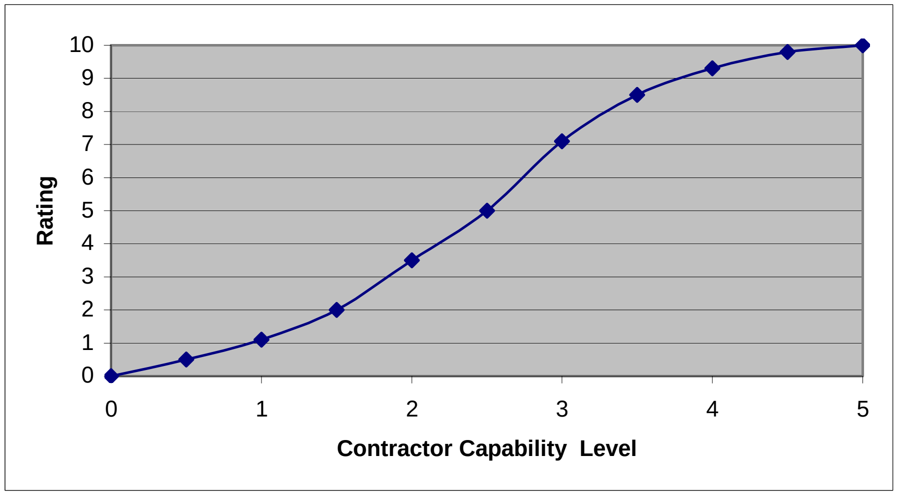


FIG. E.7. Contractor Capability for Concrete Pavement (Zollinger et al. 2001)

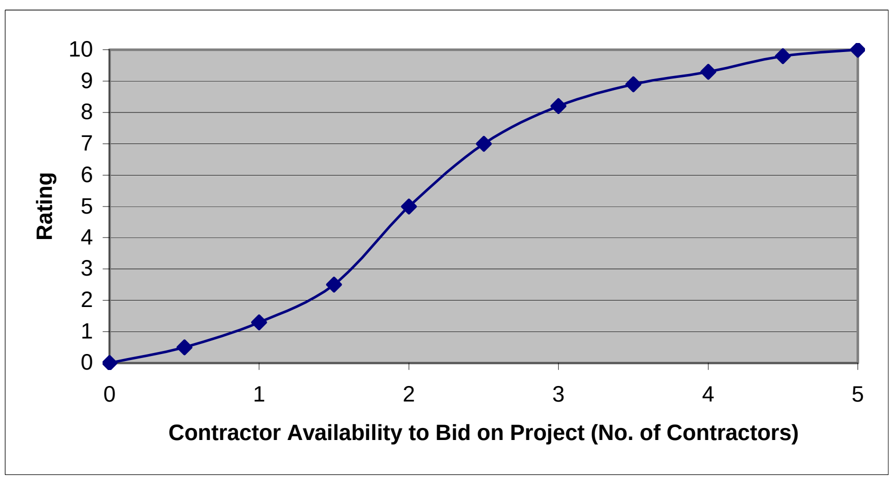


FIG. E.8. Contractor Availability for Concrete Pavement

(Zollinger et al. 2001)

Asphalt Pavement

Scale
5 - Superior
4 - Excellent
3 - Good
2 - Fair
1 - Poor
0 - None

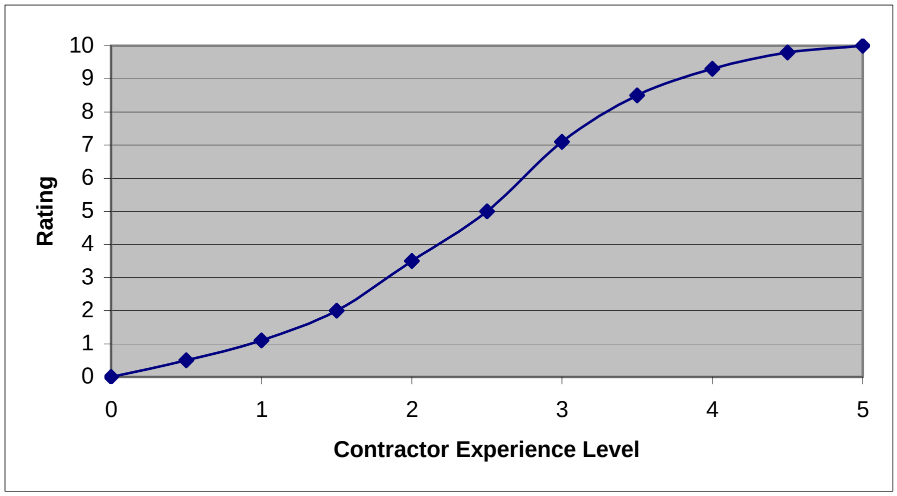


FIG. E.9. Contractor Experience for Asphalt Pavement

(Zollinger et al. 2001)

Scale
5 - Superior
4 - Excellent
3 - Good
2 - Fair
1 - Poor
0 - None

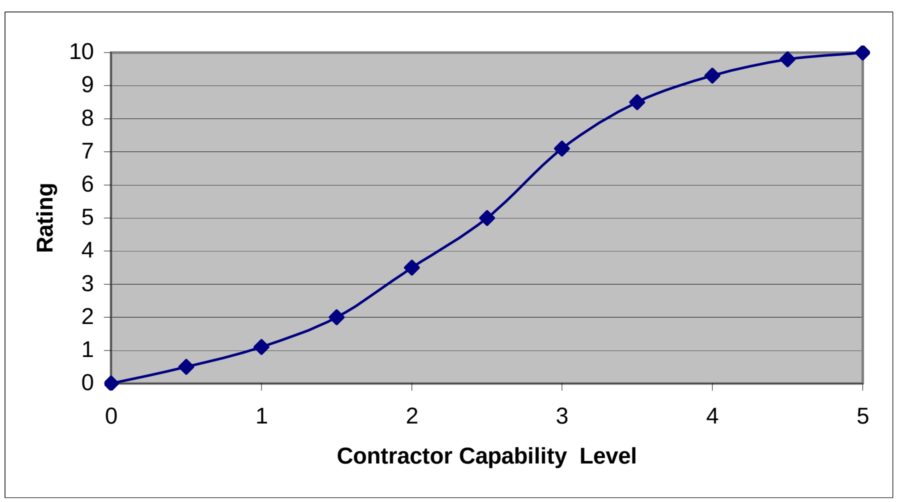


FIG. E.10. Contractor Capability for Asphalt Pavement

(Zollinger et al. 2001)

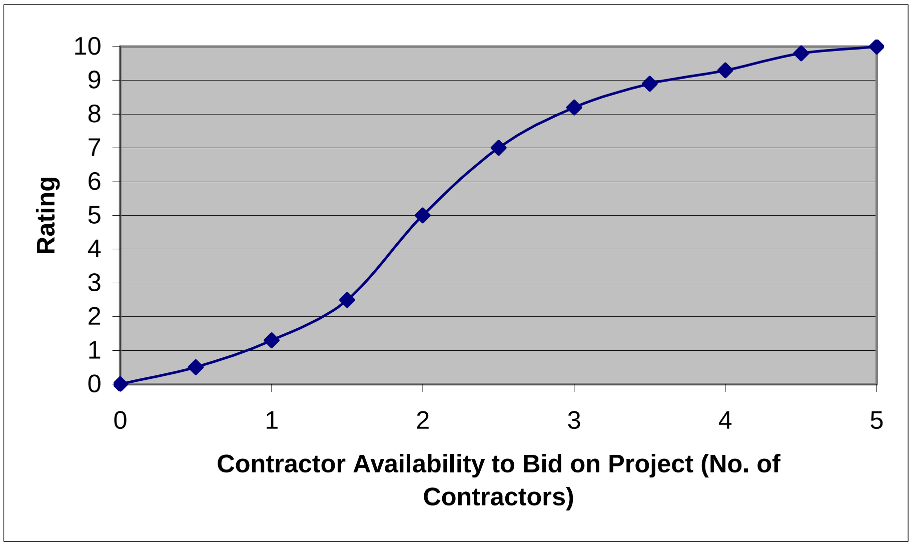


FIG. E.11. Contractor Availability for Asphalt Pavement (Zollinger et al. 2001)

APPENDIX F

EXAMPLE MRR STRATEGY SELECTION

The following example is provided to demonstrate the procedure depicted in the foregoing chapters and appendices. The project data is from an actual site in Arkansas on I-30 W near Texarkana that was documented in “Performance of Continuously Reinforced Concrete Pavements: Volume IV – Resurfacings and Rehabilitation of CRC Pavements,” Final Report, FHWA – RD – 98 – 100. Assumptions have been made were data was missing to facilitate the step-by-step procedure. Arkansas DOT placed 150 mm (6 inch) unbounded CRC over an existing 254 mm (10 inch) JRCP in late 1991. This alternative did not last long and pavement was removed and reconstructed in 1996.

Pavement Condition Data

Visual survey and non-destructive testing were carried out to evaluate the pavement condition prior to the rehabilitation. The details of pavement conditions are illustrated in Table F.1.

TABLE F.1. Pavement Condition

Condition		**Amount**	**Severity Level**
Structural Condition	Mid-slab cracking	12%	High
	Patches	70 number/mile	High
Functional Condition	Spalling	30%	Moderate
	Faulting	4 mm	High
	Ride quality	PSI 3.1	Low
	Skid number	30	Low

Task 3 Develop Feasible Strategies

Subtask 1 Develop Treatment Combinations

Using the feasibility flowchart and the current pavement condition as shown in Table F.1, following options are suggested:

- FDR + PDR + Diamond Grinding
- UCOL (w/slab stabilization + retrofit dowel bars + limited FDR)
- Rubblization with HMA overlay

Subtask 1 a Assess remaining life

In subtask 1 a pavement structural and functional evaluations were assessed using the existing pavement conditions, and the results shown in Table F.2. Each rating and weighing was determined using appropriate curves provided in appendixes C, D and E.

TABLE F.2. Rating of Existing Pavement Structural and Functional Condition

<table>
<tr><th colspan="2">Condition</th><th>Rating (0-10)</th><th colspan="2">Weight</th><th>Product</th><th>Sum</th><th>Weight</th></tr>
<tr><td rowspan="2">Structural Condition</td><td>Patches</td><td>2.5</td><td colspan="2">0.5</td><td>1.25</td><td rowspan="2">3.5</td><td rowspan="2">0.5</td></tr>
<tr><td>Slab Cracking (%)</td><td>4.5</td><td colspan="2">0.5</td><td>2.25</td></tr>
<tr><td rowspan="7">Functional Condition</td><td>Spalling Level (%)</td><td>5</td><td>0.3</td><td rowspan="3">0.3</td><td rowspan="3">1.95</td><td rowspan="7">6.5</td><td rowspan="7">0.5</td></tr>
<tr><td>Faulting (mm)</td><td>6</td><td>0.3</td></tr>
<tr><td>Ride Quality PSI</td><td>8.5</td><td>0.3</td></tr>
<tr><td>Skid Number</td><td>6.5</td><td colspan="2">0.1</td><td>0.65</td></tr>
<tr><td>Noise level (dB)</td><td>4</td><td colspan="2">0.1</td><td>0.4</td></tr>
<tr><td>Subbase Strength</td><td>7</td><td colspan="2">0.2</td><td>1.4</td></tr>
<tr><td>Subgrade Drainage</td><td>7</td><td colspan="2">0.3</td><td>2.1</td></tr>
<tr><td colspan="8">Overall Existing Condition = W_{SC}*U_{SC} + W_{FC}*U_{FC} = 3.5*0.5 + 6.5*0.5 = 5.0</td></tr>
</table>

As noted in table F.2, the distress types are slab cracking, spalling and faulting. However, remaining life was considered only for cracking and faulting distress types. Table F.3 lists the remaining life of pavement for cracking and faulting distress.

TABLE F.3. Remaining Life of Pavement

Pavement Condition	Remaining Life
Cracking	7.9 years
Faulting	4.9 years

From table F.3 relative life of pavement with respect to cracking is 7.9 years while with respect to faulting is 4.9 years. This result in the user assigning a rating of 3.9 based on obtaining the desired pavement service life of 12 years or more.

Also structural condition rating is 3.5, which is more than 5, functional condition rating is 6.5, which is more than 2.5, and remaining life rating is 3.9 which is less than 5. Based on the decision criteria outline in Table 4.1, CPR strategy can be considered.

Subtask 1b Assess Life Extension

Tables F.4, F.5 and F.6 show the rating of treated pavement condition relative to structural and functional effects of CPR, UCOL (unbounded concrete overlay) and rubblization respectively.

TABLE F.4. Rating of Pavement Structural and Functional Condition (CPR)

<table>
<tr><th colspan="2">Condition</th><th>Rating (0-10)</th><th colspan="2">Weight</th><th>Product</th><th>Sum</th></tr>
<tr><td rowspan="2">Structural Condition</td><td>Patches</td><td>9</td><td colspan="2">0.5</td><td>4.5</td><td rowspan="2">9.5</td></tr>
<tr><td>Slab Cracking (%)</td><td>10</td><td colspan="2">0.5</td><td>5</td></tr>
<tr><td rowspan="7">Functional Condition</td><td>Spalling Level (%)</td><td>9</td><td>0.3</td><td rowspan="3">0.3</td><td rowspan="3">2.8</td><td rowspan="7">7.35</td></tr>
<tr><td>Faulting (mm)</td><td>10</td><td>0.3</td></tr>
<tr><td>Ride Quality PSI</td><td>9</td><td>0.3</td></tr>
<tr><td>Skid Number</td><td>7.5</td><td colspan="2">0.1</td><td>0.75</td></tr>
<tr><td>Noise level (dB)</td><td>5</td><td colspan="2">0.1</td><td>0.5</td></tr>
<tr><td>Subbase Strength</td><td>6</td><td colspan="2">0.2</td><td>1.2</td></tr>
<tr><td>Subgrade Drainage</td><td>7</td><td colspan="2">0.3</td><td>2.1</td></tr>
<tr><td colspan="3">Overall Condition</td><td colspan="4">8.4</td></tr>
</table>

TABLE F.5. Rating of Pavement Structural and Functional Condition (UCOL)

<table>
<tr><th colspan="2">Condition</th><th>Rating (0-10)</th><th colspan="2">Weight</th><th>Product</th><th>Sum</th></tr>
<tr><td rowspan="2">Structural Condition</td><td>Patches</td><td>10</td><td colspan="2">0.5</td><td>5</td><td rowspan="2">10</td></tr>
<tr><td>Slab Cracking (%)</td><td>10</td><td colspan="2">0.5</td><td>5</td></tr>
<tr><td rowspan="7">Functional Condition</td><td>Spalling Level (%)</td><td>10</td><td>0.3</td><td rowspan="3">0.3</td><td rowspan="3">3</td><td rowspan="7">7.95</td></tr>
<tr><td>Faulting (mm)</td><td>10</td><td>0.3</td></tr>
<tr><td>Ride Quality PSI</td><td>10</td><td>0.3</td></tr>
<tr><td>Skid Number</td><td>7.5</td><td colspan="2">0.1</td><td>0.75</td></tr>
<tr><td>Noise level (dB)</td><td>7</td><td colspan="2">0.1</td><td>0.7</td></tr>
<tr><td>Subbase Strength</td><td>7</td><td colspan="2">0.2</td><td>1.4</td></tr>
<tr><td>Subgrade Drainage</td><td>7</td><td colspan="2">0.3</td><td>2.1</td></tr>
<tr><td colspan="3">Overall Condition</td><td colspan="4">9.0</td></tr>
</table>

TABLE F.6. Rating of Pavement Structural and Functional Condition (Rubblization)

<table>
<tr><th colspan="2">Condition</th><th>Rating (0-10)</th><th colspan="2">Weight</th><th>Product</th><th>Sum</th></tr>
<tr><td rowspan="2">Structural Condition</td><td>Patches</td><td>10</td><td colspan="2">0.5</td><td>5</td><td rowspan="2">10</td></tr>
<tr><td>Slab Cracking (%)</td><td>10</td><td colspan="2">0.5</td><td>5</td></tr>
<tr><td rowspan="7">Functional Condition</td><td>Spalling Level (%)</td><td>10</td><td>0.3</td><td rowspan="3">0.3</td><td rowspan="3">3</td><td rowspan="7">8.45</td></tr>
<tr><td>Faulting (mm)</td><td>10</td><td>0.3</td></tr>
<tr><td>Ride Quality PSI</td><td>10</td><td>0.3</td></tr>
<tr><td>Skid Number</td><td>7.5</td><td colspan="2">0.1</td><td>0.75</td></tr>
<tr><td>Noise level (dB)</td><td>7</td><td colspan="2">0.1</td><td>0.7</td></tr>
<tr><td>Subbase Strength</td><td>8</td><td colspan="2">0.2</td><td>1.6</td></tr>
<tr><td>Subgrade Drainage</td><td>8</td><td colspan="2">0.3</td><td>2.4</td></tr>
<tr><td colspan="3">Overall Condition</td><td colspan="4">9.2</td></tr>
</table>

Applying CPR changes the pavement cracking and faulting condition and results in a life extension relative to cracking of approximately 9.4 years and by faulting of 7 years. The life extension for UCOL and rubblization was determined to be 20 years. The life extension ratings for CPR, UCOL and Rubblization were determined to be 2.5, 10 and 10 respectively (from table 4.1 minimum LE for CRC overlay of 70% based on 0-100% scale), respectively based on a need to achieve over 12 years of performance.

Based on the calculation of SC rating, FC rating, RL rating and LE rating in subtask 1a and 1b, we can draw the conclusion that all the alternative strategies of repair pass the table 4.1 criteria.

Subtask 2 Determine Traffic Impact and Time of Construction

- The traffic condition was listed as following:
- ESAL = 12 million
- Traffic composition: mix 90% auto, 5.4% SU, 4.6% Combo
- ADT of 11,230 vehicles per day lane, tolerable construction time is 3 months
- Work zone duration:

- Work zone duration is 60 days for CPR; for UCOL and rubblization, 30 days for pretreatment and 30 days for paving
- Time of construction rating: 8 for CPR and 4 for using overlay based on user satisfaction relative to the degree of inconvenience experienced by the public.

➢ 3 lanes will remain open during the non-work zone periods (Cap. 6285 vph) and 2 lanes will be open – 1 lane closed during the work zone (Cap. 3027 vph) periods.
➢ Length = 5.25 miles
➢ Approach speed = 55 mph and work zone speed = 40 mph
➢ No problems are noted relative to cosntructibility of either repair strategy.

Subtask 3 Estimate First Cost

Table F.7 gives cost estimation that includes material, labor, equipment and contractor overheads and profit within 10 years for discounted future repairs and table F.8 shows the items included in the first cost estimation for both strategies.

TABLE F.7. Cost of Material, Labor, Equipment,

Contractor Overhead and Profit

Strategy	Item	Unit Cost	Quantity (/mile)	Cost (/mile)	Sum
CPR	Full-depth repairs	$ 60.00/SY	3200	$192,000	$1,377,600
	Diamond Grinding	$ 5.00//SY	14080	$70,400	
UCOL	9" CRC Overlay	$ 35.00/SY	4987	$174,545	$1,090,304
	2" AC Bond Breaker	$ 30.00/Ton	60	$4,928	
	Slab Stabilization	$ 1.25/SY	1760	$2,200	
	Retrofit Dowel bars	$ 72.00/Joint	352	$25,344	
	Limited FDR	$ 60.00/SY	11	$660	
Rubblization	Break and Seat	$ 0.99/SY	87991	$73,760	$1,157,171
	Fabric	$ 0.36/SY	1644400	$50.11	
	Tack Coat	$ 0.59/Gal	26100	$12,954	
	Asphalt Binder	$ 0.56/Gal	57541	$27,284	
	Type A, AC	$ 16.21/Ton	27390	$375,945	
	Type B, AC	$ 18.43/Ton	42753	$667,178	

TABLE F.8. First Cost Estimates

Cost Type	CPR	UCOL	Rubblization
Cost of material, labor, equipment, contractor overhead and profit	$1,377,600	$1, 090,304	$1,157,171
Utility and drainage modification costs	$0	$200,000	$200,000
Traffic control cost	$120,000	$80,000	$80,000
Cost of public awareness plan	$30,000	$60,000	$60,000
First cost	$1,527,600	$1,430,304	$1,497,171

Based on the user satisfaction, the first cost ratings of these two strategies were 4 for CPR 5 for Rubblization and 6 for UCOL.

Subtask 4 Identify Suitable Strategies

In table F.9, the overall rating for each strategy is determined and compared to the criteria for suitability and further considerations under task 4. The overall rating of UCOL and rubblization is much higher than CPR.

TABLE F.9. Overall Rating for Identifying Suitable Strategies

	Condition	Rating (0-10)	Weight	Product	Sum
CPR	Overall pavement rating	8.4	0.3	2.52	5.67
	Life extension rating	2.5	0.3	0.75	
	Cost rating	4	0.2	0.8	
	Time of construction rating	8	0.2	1.6	
UCOL	**Condition**	**Rating (0-10)**	**Weight**	**Product**	**Sum**
	Overall pavement rating	9	0.3	2.7	8.1
	Life extension rating	10	0.4	4	
	Cost rating	6	0.1	0.6	
	Time of construction rating	4	0.2	0.8	
Rubblization	**Condition**	**Rating (0-10)**	**Weight**	**Product**	**Sum**
	Overall pavement rating	9.2	0.3	2.76	9.16
	Life extension rating	10	0.4	4	
	Cost rating	8	0.2	1.6	
	Time of construction rating	4	0.2	0.8	

Subtask 1 Determine LCC

Tables F.10 and F.11 list the future cost for CPR, UCOL and Rubblization, the known condition for calculating LCC, respectively, assuming a discount rate of 4% for 15 year period.

TABLE F.10. Future Cost for CPR, UCOL and Rubblization

Strategy	Item	Interval	Unit cost	Quantity (/mile)	Total ($/mile)	Cost
CPR	Diamond Grinding	5	$5.00/SY	14080	$70,400	$369,600
		7	$5.00/SY	14080	$70,400	$369,600
		15	$5.00/SY	14080	$70,400	$369,600
Rubblization	Microsurfacing	10	$1.75/SY	7056	$13,053	$61,740
	Crack rout and seal	5	$1.00/LF	5264	$5,264	$7,513
UCOL	CRC longitudinal joint rout and seal	10	$ 1.00/LF	5264	$5,264	$27,636
	CRC Crack rout and seal	7	$0.8/LF	1908	$1,431	$7,513
	Slab Stabilization	15	$1.25/SY	3520	$4,400	$23,100

CPR

NPV for agency cost = *$1,527,600 + $369600(0.8219) + $369,600(0.6756) + $369,600(0.5553) = $2,285,417*

UCOL

NPV for agency cost = *$1,430,304 + $27,636(0.6756) + $7,513(0.6756) + $23,100(0.5553) = $1,466,878*

Rubblization

NPV for agency cost = *$1430304 + $61740*0.656 + $7513*0.7599 = $1476514*

The cost rating of LCC is determined based on the user satisfaction. In our example, the cost rating of LCC for CPR is 5.5 and for UCOL and Rubblization are 7.5.

Subtask 1 a Determine Non-agency Cost

Non-agency cost includes vehicle operating cost, delay cost, and accident cost. The details of calculation of first non-agency cost were listed in Table F.11. These costs are determined based on CPR being done at night as well as the pretreatment work for the UCOL where the lanes are open during daytime hours. However, paving of UCOL and rubblization are assumed to be done in the daytime over 4-week period under lane closure.

TABLE F.11. First Non-Agency Costs

Cost Component		CPR	UCOL	Rubblization
Vehicle Operating Cost	WZ Speed Change VOC (55-40-55 mph)	$9,809	$9,809	$9,809
	Queue Stopping VOC (55-0-55 mph)	$5,785	$73,103	$73,103
	Queue Idle VOC	$30,204	$381,993	$381,993
Delay Cost	WZ Speed Change Delay (55-40-55 mph)	$6,393	$6,393	$6,393
	Queue Stopping Delay (55-0-55 mph)	$4,459	$56,356	$56,356
	WZ Reduced Speed Delay (40 vs. 55 mph)	$130,685	$231,867	$231,867
	Queue Added Travel Time Delay (8 vs.55 mph)	$535,071	$6,766,250	$6,766,250
Accident Cost		$15,227	$44,500	$44,500
Total First Cost		$737,634	$7,570,273	$7,570,273

Subtask 2 Assess Corridor Impact

The details for corridor impact assessment are shown in tables F.12, F.13 and F.14 respectively.

TABLE F.12. Corridor Impact of CPR

Corridor Impact	Rating	Weight	Product
Noise Level	8	0.1	0.8
Air Pollution Level	7	0.1	0.7
Sum	1.5		

TABLE F.13. Corridor Impact of UCOL

Corridor Impact	Rating	Weight	Product
Noise Level	5	0.1	0.5
Air Pollution Level	6	0.1	0.5
Sum	1.1		

TABLE F.14. Corridor Impact of Rubblization

Corridor Impact	Rating	Weight	Product
Noise Level	6	0.1	0.6
Air Pollution Level	9	0.1	0.7
Sum	1.5		

Subtask 3 Assess Constructability

Tables F.15, F.16 and F.17 show the assessment of constructibility for the three strategies.

TABLE F. 15. Constructibility of CPR

Constructibility	Rating	Weight	Product
Contractor Experience, Capability and Availability	8	0.4	3.2
Future Limitations	6	0.3	1.8
WZ safety	6	0.2	1.2
Sum	6.2		

TABLE F.16. Constructibility of UCOL

Constructibility	Rating	Weight	Product
Contractor Experience, Capability and Availability	7	0.4	2.8
Future Limitations	4	0.3	1.2
WZ safety	7	0.2	1.4
Sum	5.4		

TABLE F. 17. Constructibility of Rubblization

Constructibility	Rating	Weight	Product
Contractor Experience, Capability and Availability	8	0.4	3.2
Future Limitations	4	0.3	1.2
WZ safety	8	0.2	1.6
Sum	6		

Subtask 4 Evaluate Feasible Strategies

Table F.18, F.19 and F. 20 show the evaluation of all the three strategies and then outlines the most preferred strategy.

TABLE F.18. Evaluation of CPR

Evaluation Attribute	Rating	Weight	Product	Sum
Life Cycle Cost	5.5	0.35	1.925	5.215
Non-agency Cost	7	0.25	1.75	
Corridor Impact	1.5	0.2	0.3	
Constructibility	6.2	0.2	1.24	

TABLE F.19. Evaluation of UCOL

Evaluation Attribute	Rating	Weight	Product	Sum
Life Cycle Cost	7.5	0.35	2.625	5.425
Non-agency Cost	6	0.25	1.5	
Corridor Impact	1.1	0.2	0.22	
Constructibility	5.4	0.2	1.08	

TABLE F.20. Evaluation of Rubblization

Evaluation Attribute	Rating	Weight	Product	Sum
Life Cycle Cost	8	0.35	2.8	5.55
Non-agency Cost	5	0.25	1.25	
Corridor Impact	1.5	0.2	0.3	
Constructibility	6	0.2	1.2	

Since the rating of Rubblization is more than CPR and UCOL, rubblization is the then selected as the preferred strategy.

VITA

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