Rehabilitation of Concrete Pavements Utilizing Rubblization and Crack-and-Seat Methods: Phase II - Performance Evaluation of Rubblized Pavements in Iowa

National Concrete Pavement Technology Center

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Final Report
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Center for Transportation Research and Education

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16. Abstract

This Phase II follow-up study of IHRB Project TR-473 focused on the performance evaluation of rubblized pavements in Iowa. The primary objective of this study was to evaluate the structural condition of existing rubblized concrete pavements across Iowa through Falling Weight Deflectometer (FWD) tests, Dynamic Cone Penetrometer (DCP) tests, visual pavement distress surveys, etc. Through backcalculation of FWD deflection data using the ISU's advanced layer moduli backcalculation program, the rubblized layer moduli were determined for various projects and compared with each other for correlating with the long-term pavement performance. The AASHTO structural layer coefficient for rubblized layer was also calculated using the rubblized layer moduli. To validate the mechanistic-empirical (M-E) hot mix asphalt (HMA) overlay thickness design procedure developed during the Phase I study, the actual HMA overlay thicknesses from the rubblization projects were compared with the predicted thicknesses obtained from the design software.

The results of this study show that rubblization is a valid option to use in Iowa in the rehabilitation of PCC provided the foundation is strong enough to support construction operations during the rubblization process. The M-E structural design methodology developed during Phase I can estimate the HMA overlay thickness reasonably well to achieve long-lasting performance of HMA pavements. The rehabilitation strategy is recommended for continued use in Iowa under those conditions conducive for rubblization.

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PERFORMANCE EVALUATION OF RUBBLIZED PAVEMENTS IN IOWA

Final Report
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EXECUTIVE SUMMARY

The primary objective of this research project was to evaluate the structural condition of existing rubblized concrete pavements in Iowa. The research study included a review of existing literature as well as experimental tests on and analyses of existing rubblized pavements across Iowa through Falling Weight Deflectometer (FWD) tests, Dynamic Cone Penetrometer (DCP) tests, visual pavement distress surveys, etc. Results of the analyses show that Iowa's rubblized pavement sections are performing very well. The predominant distresses exhibited on hot mix asphalt (HMA) overlaid rubblized portland cement concrete (PCC) sections are non-load-associated distresses, such as low temperature cracking and/or longitudinal cracking. The average PCC modulus of the rubblized layer was found to be 78 ksi, which is close to the modulus value of 65 ksi recommended by a Wisconsin Department of Transportation (DOT) study. The average rubblized PCC American Association of State and Highway Transportation Officials (AASHTO) layer coefficient value in this study was found to be 0.19, which is consistent with that used by Arkansas, Michigan, Mississippi, Ohio, and Pennsylvania. The average tensile strain value of 74 $\mu\varepsilon$ (microstrain) at the bottom of HMA layer and the average vertical strain of 235 $\mu\varepsilon$ at the top of subgrade are close to 70 $\mu\varepsilon$ and 200 $\mu\varepsilon$ required for long-lasting HMA pavements. In addition, the analyses verified that the M-E design methodology developed during Phase I of this research can estimate the HMA overlay thickness reasonably well to achieve long-lasting performance of HMA overlay pavements with rubblization. It is recommended that Iowa DOT continue to use PCC rubblization as a valid pavement rehabilitation strategy.

INTRODUCTION

Background

An asphalt overlay of a fractured concrete pavement is placed to increase the structural capacity of the pavement. Slab fracturing may be done for two reasons: to attempt to mitigate reflection cracking in the overlay, and/or to dispense with pre-overlay repair of a concrete pavement with extensive cracking and/or materials-related deterioration (e.g., “D” cracking, alkali-silica reaction, alkali-carbonate reaction, etc.). Several surface preparation techniques have been used before placing an HMA overlay in attempts to minimize reflection cracking. Some of the most common techniques are rubblization, crack-and-seat, break-and-seat, and saw-and-seal (Hall et al. 2001).

Thompson’s 1989 National Cooperative Highway Research Program (NCHRP) Synthesis of Highway Practice summarized breaking/cracking/seating (B/C/S) practices and technology. Several field investigative studies have indicated that the B/C/S techniques delayed, but did not eliminate reflection cracking (Thompson 1999). The results from a comprehensive investigation conducted by PCS/Law (1991), the National Asphalt Pavement Association (NAPA) study (NAPA 1994), and a nationwide survey conducted by the Florida DOT (Ksaibati et al. 1999) all indicate that rubblization is the most effective procedure for addressing reflection cracking. Rubblization involves breaking the existing PCC slab into pieces (usually ranging from 2 to 6 inches) and overlaying with HMA. It has been concluded that the rubblized PCC behaves like “a high-strength granular base,” with strength between 1.5 to 3 times greater than a high-quality, dense-graded, crushed-stone base in load-distributing characteristics (PCS/Law 1991).

The design of the structural overlay thickness for rubblized projects is difficult, as the resulting structure is neither a “true” rigid pavement nor a “true” flexible pavement. Classical rigid pavement analysis and design is based upon the Westergaard theory, while classical flexible pavement analysis and design is based upon the Burmister multi-layer theory. Based on the assumption that the rigidity of the PCC slabs has been destroyed, the Burmister approach may be used with HMA-overlaid fractured PCC pavement. It has been proposed that the Westergaard approach may be used to evaluate the pre-rubblized PCC slabs, whereas Burmister theory may be used for post-rubblization analysis (Bemanian and Sebaaly, 1999).

HMA overlay thickness design procedures for rubblized PCC pavements have been proposed by NAPA and the Asphalt Institute, based on the structural number-layer coefficient principles used in the existing 1993 AASHTO design guide. The AASHTO design guide requires the determination of a layer coefficient for the rubblized PCC. This coefficient varies considerably depending on the state agency and the design procedure used, giving rise to a wide range of HMA overlay thicknesses. As a result of the analysis of 19 existing sections, layer coefficient values in the practical range of 0.23 to 0.31 were recommended by PCS/Law in a report dated June 1991. These recommendations are based on results from different states, and they therefore reflect differences in material specifications and construction practices. Thus, there is a need to estimate the in situ layer coefficient of rubblized concrete pavements in Iowa to provide recommendations for future design.

Problem Statement

In Iowa, a significant portion of surfaced highway pavements are of the PCC type. These pavements deteriorate over time due to distresses caused by a combination of traffic loads and weather conditions. There are various PCC pavement rehabilitation alternatives, including bonded and unbonded concrete overlays, full-depth repair, partial-depth repair, joint and crack repairs, asphalt or concrete overlay of fractured concrete slab, etc. (Hall et al. 2001). Among these, the most common form of PCC pavement rehabilitation is overlaying with HMA (PCS/Law 1991; Freeman 2002). However, experience has shown that in order to prevent the occurrence of reflective cracking from joints and cracks in the PCC pavement and the reflection of other PCC distresses into the overlaying HMA layers, it is necessary to destroy the slab action of the PCC slabs (Freeman 2002).

Over the past 15 years, the fractured slab techniques, which seek to destroy the slab action by reducing the slab into smaller component sizes, have gained increased acceptance as a means of retarding the formation of reflection cracks. Among the fractured slab techniques, the break-and-seat technique is generally applicable to jointed reinforced concrete pavements (JRCP) and the crack-and-seat is used for rehabilitation of jointed plain concrete pavements (JPCP), while rubblization can be recommended for any type of deteriorated PCC pavement (PCS/Law 1991). While these methods have proven to be generally successful in transforming a piece-wise rigid structure into a flexible or quasi-flexible structure, the thickness design of the HMA overlay in terms of design life are not well known, nor understood.

During the first phase of the Iowa Highway Research Board (IHRB) Project TR-473, "Rehabilitation of Concrete Pavements Utilizing Rubblization and Crack and Seat Methods," which was completed recently, researchers studied the effects of PCC rubblization and crack-and-seat operations on the HMA overlay thickness necessary to achieve the desired design life (Ceylan et al. 2005). The objective was to develop a design procedure for determining the HMA overlay thickness for rubblized PCC pavements. A partial validation of the design approach was provided with reference to an instrumented trial project (rubblized pavement) in Iowa (Highway IA-141, Polk County).

The asphalt strain gage measurements from the trial sections and the design model strain predictions showed that strain values were less than or close to 70 microstrain, even in the peak summer period when high pavement temperatures exist. Research studies suggest that, for long-lasting HMA pavements, the limiting tensile strain at the bottom of the asphalt concrete layers should be no greater than 70 microstrain, and that at the top of the subgrade the vertical strain should be limited to 200 microstrain (TRB 2001). The results from the Phase I study indicated that a 9-inch HMA overlay thickness may not be required if long-lasting performance could be achieved from a 7.5-inch HMA overlay thickness based on the field-measured strain values. Thus, the developed design procedure has the potential to predict the minimum HMA overlay thickness required, resulting in significant cost savings.

The final report recommended that the performance of the instrumented trial project and other rubblized pavements be continuously monitored for validation of the design procedure developed during Phase I. More structural condition assessment data from existing rubblized pavements is required to validate the HMA overlay thickness design procedure and provide layer coefficient recommendations for future design of rubblized concrete pavements.

Objectives

This study was proposed as a Phase II follow-up study of IHRB Project TR-473. Phase I of this project focused on developing a structural design procedure for the design of HMA overlay thickness for rubblized concrete pavements. Based on the recommendations of the Phase I study, the Phase II study focused on validating the developed design procedure by evaluating the performance of in-service rubblized pavements in Iowa.

The primary objective of this study is to evaluate the structural condition of existing rubblized concrete pavements across Iowa through Falling Weight Deflectometer (FWD) tests, Dynamic Cone Penetrometer (DCP) tests, visual pavement distress surveys, etc. Through backcalculation of FWD deflection data, the rubblized layer modulus values were determined for various projects and compared with each other for correlating with long-term pavement performance. The results would be useful in establishing design modulus and for providing AASHTO layer coefficient recommendations for rubblized PCC layers.

The following are the secondary objectives of this study:

1. To examine design, construction, and performance records of existing overlaid fractured PCC pavements to estimate the effects of subgrade, fractured slab thickness and structural value, and overlay thickness on performance
2. To develop a knowledge database based on the findings of the pavement performance study, which will be useful in the selection of cost-effective PCC pavement rehabilitation strategies in Iowa

OVERVIEW OF RUBBLIZATION TECHIQUES AND EXPERINCES

Fractured slab techniques include rubblization, break-and-seat, and crack-and-seat. They have been used for many years in the rehabilitation of pavements. The primary goal of these techniques is to reduce the reflective cracking in the HMA overlay by reducing the concrete slabs to pieces. The results obtained from the investigation of PCS/Law in 1991 indicate that rubblization is the best rehabilitation technique, followed by crack-and-seat, saw-and-seal, and then break-and-seat. Considerable research has been conducted on rubblization in the recent past, and studies indicate that the performance of this technique varies from place to place and from project to project. The variation is due to factors such as the condition of the existing PCC pavement, type and level of distress, type of construction equipment, environmental conditions, traffic, and type and thickness of HMA overlay.

Rubblization techniques and usage are summarized in this chapter. Current states' practices on the use of rubblization and their performance experiences are also reviewed and provided in this chapter.

Rubblization Techniques

Rubblization of deteriorated concrete pavements followed by HMA overlay is an excellent rehabilitation method that is equally effective for all types of PCC pavements. Rubblization is defined as "breaking the existing pavement into pieces and overlaying with HMA." It destroys the slab action of the rigid pavements. The sizes of the broken pieces usually range from sand size to 3 in. at the surface and from 12 to 15 in. on the bottom part of the rubblized layer (Von Quintus et al. 2007). The rubblized PCC pavement behaves like a high-quality granular base layer, the rubblized layer responds as an interlocked unbound layer—reducing the existing PCC to a material comparable to a high-quality aggregate base course. This loss of structure must be accounted for in the HMA overlay design thickness (Galal et al. 1999). A study by NAPA indicated that strength of the rubblized layer is 1.5 to 3 times greater than a high-quality dense graded crushed stone base (PCS/Law 1991). The fractured slab eliminates reflective cracking in HMA overlays by minimizing thermal expansion and contraction of the PCC slabs.

In general, two types of equipment are used in the rubblization process: Resonant Pavement Breaker (RPB) and Multiple-Head Breaker (MHB). The rubblization procedure plays an important role in long-term performance of the pavement. The RPB shown in Figure 1 uses vibrating hammers to demolish the existing pavement. This system breaks the concrete slab and destroys the bond between the concrete and the steel. It works on the principle that the frequency of a vibratory force can be varied until the resonant frequency of the body being vibrated can be determined. More details of the equipment are available at the company's website (http://www.resonantmachines.com/).

The other common rubblizing equipment is the MHB used by Antigo Construction, shown in Figure 2. The equipment is of the self-contained and self-propelled type, which is capable of rubblizing the pavement over a minimum width of 13 ft. per pass. The hammers used by this breaker are mounted laterally in pairs, with half of the hammers in the forward row and the remainder diagonally offset in the rear row. More details of this equipment are available at Antigo's website (http://www.antigoconstruction.com/).

Figure 1. Resonant Pavement Breaker (Resonant Machines, Inc.)

Figure 2. Multi-Head Breaker (Antigo Construction, Inc.)

Rubblization Usage

The Asphalt Institute recently reported that more than 50 million square yards of U.S. highways were successfully rubblized between 1994 and 2002, beginning with the first project in New York in 1986 (Von Quintus et al. 2007). Table 1 lists those agencies and the approximate number of rehabilitation projects for which the rubblization process has been used. This technique also has been implemented in Canada, Russia, Yugoslavia, Chile, and China (Von Quintus et al. 2007). Other government agencies, such as the Federal Aviation Administration (FAA), the U.S. Air Force, and the U.S. Army Corps of Engineers, have identified the rubblization process as a viable technique and are developing design and construction guidance for HMA overlays of rubblized airport pavements.

Table 1. Relative level of use of the rubblization process by state agencies (Von Quintus et al. 2007)

Level of Use	State Agency	Approximate Number of Projects
Heavy Use of Rubblization; > 20 Major Rehabilitation Projects	Alabama	20+
	Arkansas	40+
	Indiana	30+
	Michigan	60+
	New York	30+
	Wisconsin	70+
	Illinois	20+
Moderate Use of Rubblization; > 5 Major Rehabilitation Projects	Florida	5+
	Iowa	10+
	Kentucky	5+
	Louisiana	10+
	Minnesota	5+
	Mississippi	5+
	Nevada	5+
	North Carolina	10+
	Pennsylvania	10+
	Ohio	10+
Limited Use of Rubblization; <5 Major Rehabilitation Projects	Colorado; Connecticut; Idaho; Kansas; Maryland; Massachusetts; Missouri; New Hampshire; New Jersey; Oklahoma; Oregon; South Carolina; Tennessee; Texas; Vermont; Virginia; Washington; West Virginia; Wyoming	1-4

Current States' Practices

HMA Overlay Thickness Design for Rubblized Concrete Pavements

The most important aspect of the design procedure is to characterize the rubblized concrete layer. The biggest challenge in the structural design of rubblized pavements is to determine the appropriate HMA overlay thickness, which satisfies both the functional and structural requirements of the pavement. Several approaches have been proposed to establish the required overlay thickness (Ceylan et al. 2005). The most recognized overlay design procedure is the 1993 AASHTO Guide for Design of Pavement Structures. It uses an empirical procedure, based upon the results of the AASHO Road Test. The 1993 AASHTO Design method uses the layer coefficient to characterize each layer. Many agencies are also considering the use of mechanistic design procedures, such as the Mechanistic-Empirical Pavement Design Guide (MEPDG) developed under NCHRP 1-37A (2004) for overlay design procedures. The elastic modulus for the rubblized layer is an important design value in MEPDG to determine the thickness of the HMA overlay. Even though the use of the layer coefficients with the 1993 AASHTO Overlay Design procedure and elastic modulus values with MEPDG have not been adequately validated with performance data (Von Quintus et al. 2007), those values reported by previous studies are presented herein.

1993 AASHTO Overlay Design Method

To calculate the overlay thickness, both the AASHTO condition survey and the nondestructive testing methods are used. Both of these methods use the concept of structural number (SN). Structural number was developed to provide a single number to represent a "conceptual" pavement thickness. The SN comprises the structural contribution of each layer, using a layer coefficient and thickness, where the layer coefficient is a measure of relative stiffness ($SN = a_1t_1 + a_2t_2 + a_3t_3 + \ldots$). Therefore, the AASHTO pavement design procedure requires estimating the layer coefficients.

According to 1993 AASHTO guide, the recommended coefficients for rubblized PCC pavement range from 0.14 to 0.30 (Thompson 1999). A study conducted by the Indiana DOT indicated that a layer coefficient of 0.22 represented a conservative value to ensure structural adequacy with similar conditions (Galal et al. 1999). The Ohio DOT uses a layer coefficient value of 0.14 to represent the rubblized layer, neglecting any existing subbase under the PCC (Von Quintus et al. 2007). Similarly, Arkansas, Michigan, Mississippi, and Pennsylvania use values between 0.14 and 0.20. Minnesota, New York, and Wisconsin have used structural layer coefficients of around 0.25. No agency has published or correlated these structural layer coefficients of the rubblized layer to different rubblization equipment and particle size distribution (Von Quintus et al. 2007).

Mechanistic-Empirical Pavement Design Guide (MEPDG)

The MEPDG requires the elastic modulus of the rubblized PCC layer to predict the performance of HMA overlay pavement. NAPA (1994) reported and recommended modulus values of 100 and 150 ksi for use in design. The default value recommended for use in the MEPDG Level 3 analysis is 150 ksi, which is a quite conservative value (2004). The study by Indiana DOT reported that the PCC modulus value before rubblization was approximately 3,800 ksi, and the rubblized PCC modulus value was about 170 ksi from US-41 in Benton County, Indiana (Galal et al. 1999). A recent study in Wisconsin reported that the average elastic modulus determined for the rubblized PCC layer in Wisconsin rubblization projects is 65 ksi and in general, the elastic modulus ranged from 35 to 120 ksi (Von Quintus et al. 2007). These values are similar to those values determined from deflection basin testing of HMA overlays placed over rubblized PCC pavements—both from the Long-Term Pavement Performance (LTPP) Specific Pavement Studies-6 (SPS-6) experiment and actual construction projects reported by Von Quintus et al. (2000).

In summary, no consistent elastic modulus value has been used to represent rubblized PCC layers, which suggests that the value is site specific and dependent on the rubblization process itself.

HMA Overlay Thickness

Table 2 summarizes the thicknesses of asphalt layers on top of the rubblized concrete as applied by various states, from a nationwide survey conducted by the Florida DOT (Ksaibati et al. 1998).

Table 2. Thickness of HMA layers on top of rubblized concrete (Ksaibati et al. 1998)

THICKNESS OF ASPHALT OVERLAY	STATES	# OF STATES*
76.2 - 152.4 mm (3" - 6")	AR, LA, ND, NV, MI, NC, NY, OK, WI, IL	10
177.8 - 254 mm (7" - 10")	AL, IL, IN, KS, MA, MD, MI, MN, MO, MS, NC, OH, OK, VA	14
279.4 - 762 mm (11" - 13")	AL, IN, MI, MO, OH, PA	6

Rubblized Pavement Construction

The Asphalt Institute has established a seven-part process for rubblized pavements that is outlined below (Fitts 2001):

1. Remove any existing overlay.
2. Install an edge drainage system, preferably two weeks before fracturing the concrete.
3. Sawcut the full thickness of the PCC pavement, along the longitudinal joint, if the adjacent pavement is to remain intact.
4. Rubblize the PCC pavement.
5. Cut and remove exposed reinforcement.
6. Roll fractured PCC.
7. Place HMA.

The Illinois DOT specifies that the construction process begin with the installation of drainage elements, as required, and getting the surface prepared. The first consideration in Illinois DOT guidelines for rubblizing concrete pavements (Heckel 2002) is whether the rubblized pavement will protect the subgrade. If conditions exist that would result in extensive removal and replacement of the existing pavement, or if the subgrade is weak and would result in severe construction problems, Illinois DOT guidelines recommend the consideration of other rehabilitation options. Wisconsin DOT (2007) also requires an investigation of subgrade strength, since construction practices consist of paving concrete pavements directly on top of subgrade, and "weak" subgrades make rubblization susceptible to subgrade yielding problems.

According to the Illinois DOT procedure, an initial evaluation is made based on soils map data and personnel experience with the soils in the proposed project limits. If this analysis indicates the pavement can be rubblized, Illinois DOT performs an extensive field analysis including, among other things, a review of the pavement structure, FWD testing, DCP testing, soil sampling, and a drainage survey. After conducting testing, Illinois DOT plots the Immediate Bearing Values (IBVs) of the top 12 inches of subgrade soil (divided into two 6-inch layers) on their Subgrade Rubblizing Guide, shown in Figure 3, to aid in deciding whether the structure can be rubblized without subgrade failure. The four methods identified for rubblization by Illinois DOT are as follows:

- Method I: use the MHB.
- Method II: use the resonant breaker with high flotation tires with a tire pressure less than 60 psi.
- Method III: use the resonant breaker without restriction on tire pressure.
- Method IV: use either the MHB or the resonant breaker.

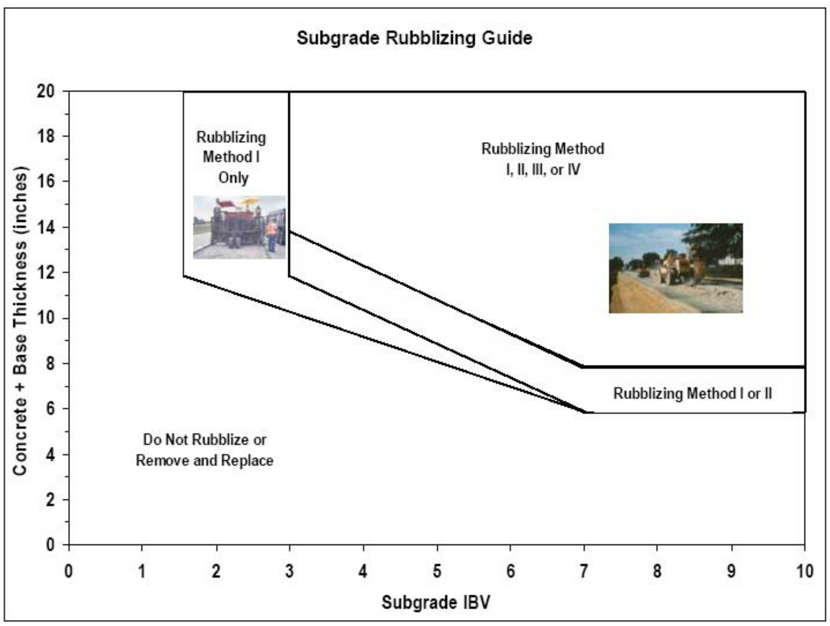

Figure 3. Illinois DOT Subgrade Rubblizing Guide (Heckel 2002)

Illinois DOT also specifies the compaction sequence on the fractured concrete layer after rubblization is completed.

After breaking:

- Minimum of four passes with Z-pattern steel grid roller (only with the MHB).

- Four passes with a vibratory roller.
- Two passes with a pneumatic-tired roller.

Immediately prior to overlay:

- Two passes with a vibratory roller.

Several DOTs, including Arkansas (2003), Michigan (2003), Ohio (2005), Illinois (Heckel 2002), and Alabama (2006) require the use of test strips and closely monitor them to calibrate the rubblization equipment to the existing site conditions. The 3 ft. by 3 ft. pit to physically observe the performance of the equipment confirms or denies the required particle sizes that are obtained at the bottom of the PCC pavement layer. The Illinois (Hecke 2002), Indiana (2005), and Ohio (2005) DOTs require that the HMA overlay is placed on the rubblized concrete within 48 hours after the rubblization process. In the event of rain, the contractor is to delay overlay placement to provide sufficient time for the moisture to drain out or dry. The rubblization process is to be discontinued in the event of rain until the paving operation starts. Additionally, no traffic is allowed to drive on the pavement until the first lift of the overlay is placed. The maximum acceptable particle sizes specified in various states are summarized in Table 3 (Ksaibati et al. 1998).

Table 3. The maximum acceptable particle sizes (Ksaibati et al. 1998)

MAXIMUM PARTICLE SIZE	STATES	# OF STATES
127 - 203.2 mm (5" - 8")	AL, IN, MI, MA, MO, MS, NC, ND, OH, OK, VA,	11
228.6 - 381 mm (9" - 15")	AR, IL, KS, LA, MD, MN, NV, NY, PA, WI	10

Rubblization Performance Experiences

National Studies

The first nationwide performance comparison of the various methods of fracturing PCC slabs was conducted by Witczak and Rada (1992). A comparison, shown in Figure 4, was developed, showing the variation in backcalculated moduli values after each of the fracturing methods. This figure shows that the moduli for the break-and-seat and crack-and-seat techniques are substantially more variable than for rubblization. This variability may be associated with the problem of fracturing slabs containing reinforcing steel. In some of the earlier projects, the steel may not be debonded from the concrete and it may still be acting as an intact slab.

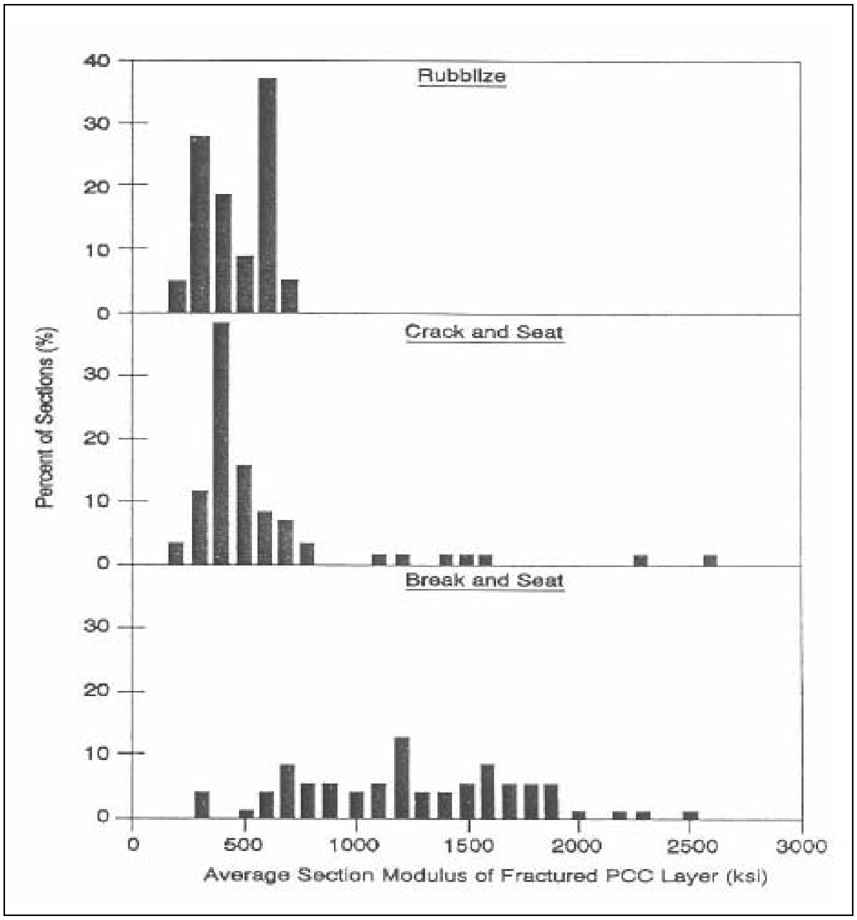

Figure 4. Frequency distribution of in-situ PCC modulus values after treatment (Witczak and Rada 1992)

The Strategic Highway Research Program (SHRP) LTPP SPS-6 experiment, "Rehabilitation of Jointed Portland Cement Concrete Pavements," is one of the key experiments of the LTPP program. The main objective of this experiment was to determine the effects of specific rehabilitation design features that directly influence the long-term effectiveness of rehabilitated JPCP and JRCP (Ambroz and Darter 2005). However, only a few of these SPS-6 projects were actually included the rubblization process. Those projects with rubblization test sections included Alabama, Arizona, Illinois, Michigan, Missouri, Oklahoma, and Pennsylvania. The test sections are listed in Table 4, which was originally made by Von Quintus et al. (2007). Some of the rubblized test sections had construction-related problems: soft foundations and non-uniform particle size distribution throughout the PCC slab thickness.

Table 4. LTPP SPS-6 projects that include rubblized test sections (Von Quintus et al. 2007)

Project-Agency	Rehabilitation Date	Test Section Identification	HMA Overlay Thickness, mm.	Comment
Alabama	6-98	0661	102	Badger Breaker Machine (Model MHB); particles down to 3 inches in size.
		0662	203	
		0663	241	
Arizona	10-90	0616	140	
		0619	140	
Illinois	6-90	0663	152	High frequency breaking unit; less than 6 inches in size; edge drains placed.
		0664	203	
Michigan	5-90	0659	178	
Missouri	8-92	0661	290	Edge drains placed.
		0662	185	
		0663	292	No edge drains placed.
		0664	175	
Oklahoma	8-92	0607	114	Resonant Frequency Breaker; surface – 2 to 3 inches in size; bottom – up to 8 inches in size; edge drains placed.
		0608	201	
Pennsylvania	9-92	0660	241	Edge drains placed.
		0661	330	

FHWA-LTPP sponsored a study in 2005 to complete an initial evaluation of the SPS-6 experiment (Ambroz and Darter 2005). The data included in the LTPP database were used to compare the performance of the different test sections. From this study, the HMA overlay of fractured PCC by rubblization and break/crack-and-seat methods were found to have a low International Roughness Index (IRI) (typically 63.36 inches/mi) immediately after rehabilitation and the lowest rate of increase in IRI after rehabilitation than any of the other rehabilitation alternatives. The fatigue cracks were found to be minimal. The direct comparison of transverse cracking (reflection cracking) between nonfractured and fractured sections at each SPS-6 site has not been determined in this study because these sections are still relatively young.

Von Quintus et al. (2007) reviewed the 2005 LTPP database to determine the current performance trends of these sections. The fatigue cracking is still considered minimal and the IRI values are low. Figure 5 shows the amount of cracking recorded in the LTPP database from the last distress survey as a function of HMA overlay thickness. In general, thicker overlays led to a lower amount of cracking, with the exception for longitudinal cracking outside wheel path. The predominant distress exhibited along these test sections is longitudinal cracking outside the wheel path area. The sections without edge drains or those with rubblized pieces less than two inches in size have higher levels of cracking.

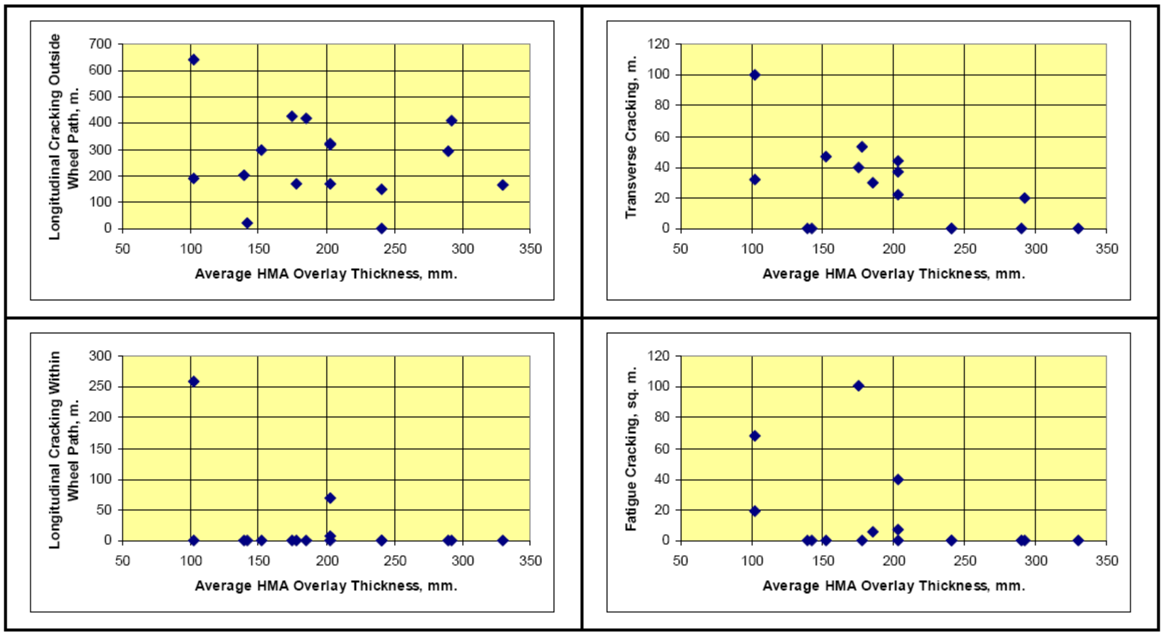

Figure 5. Amount of cracking on the LTPP SPS-6 rubblized test sections (Von Quintus et al. 2007)

An evaluation of the performance of several rehabilitation options within the SPS-6 project along I-80 in Pennsylvania was conducted at the end of 10 years of service (Morian et al. 2003). The Pennsylvania SPS-6 test sections that received no overlays have deteriorated substantially over the first ten years, while the test sections that were rubblized or cracked-and-seated and overlaid with the thicker HMA have exhibited the best performance. The rubblized sections exhibited the best structural performance and strongest subgrade support response during the first five years after rehabilitation operations. In addition, the rubblized sections have the highest reliability built into the design and their performance reflects this fact. From a performance analysis to assess the cost effectiveness of each rehabilitation option, the rubblized sections had the highest rating amongst the other options, followed by break-and-seat with third point cracks.

The Asphalt Institute conducted an evaluation of various rubblization projects across the U.S. (Fitts 2001). Results from this study found an expected service life of 22 years to a 20-year design life based on extrapolations of performance data. Although many of the projects surveyed and evaluated have exhibited little to no structural distress, the predominant distresses observed on those projects showing early signs of deterioration are longitudinal and transverse cracking. Rutting and fatigue cracking have been found to be minimal on rubblized PCC pavements. Forensic studies completed on those projects with the higher levels of cracking found that the structural integrity of the PCC slabs was not completely destroyed, which resulted in reflection cracks at the joints and cracks in the existing pavement.

Illinois (Heckel 2002; Wienrank and Lippert 2006)

The Illinois DOT was one of the first state DOTs to consider rubblization as a rehabilitation strategy, and has been using this technology since 1990 (Heckel 2002; Wienrank and Lippert

2006). To date, 12 projects using the rubblization method have been constructed on state-maintained routes in Illinois, seven of which incorporated experimental features. Five of these seven projects are located on heavily traveled Interstate routes. After about 10 years of placing HMA overlays on rubblized PCC slabs, the Illinois DOT conducted a thorough evaluation of seven experimental projects to refine its standards and guidelines for designing and constructing rubblized pavements (Heckel 2002). The performance evaluation was conducted in 2002 and included projects built between 1990 and 1999. The Illinois DOT found that performance has been very good thus far, with less reflective cracking than on adjacent patch and overlay sections, and none of the projects have required rehabilitation at this point. A recent project, constructed in 2003, was the first rubblizing project designed using a 30-year design period (Wienrank and Lippert 2006). Although this project does not contain experimental features, it is a unique design that will also be monitored closely. Rubblization is not yet a standard rehabilitation practice included in Illinois DOT's pavement selection process. Rather, it remains a special design that requires approval from Illinois DOT's Bureaus of Design and Environment and Materials and Physical Research (Wienrank and Lippert 2006). However, the Illinois DOT concluded that rubblization is the most viable option when patching quantities exceed 10% to 15%. It was suggested that the main factors to consider in project selection are the condition of the existing pavement and subgrade strength.

Indiana (Gulen et al. 2004)

Construction of HMA overlays on old concrete pavements is the most common concrete pavement rehabilitation strategy for Indiana's highway network. The Indiana DOT recently evaluated different concrete pavement rehabilitation techniques employed on interstate highway I-65 (Gulen et al. 2004). The existing pavement—a 10-in. JRCP on 8-in. sandy subbase—was built in 1968 and restored in 1985 without overlay. Three rehabilitation techniques were employed for rehabilitation of I-65 pavement segments during the years 1993-1994:

- a fiber modified HMA overlay on cracked-and-seated concrete pavement,
- an HMA overlay on rubblized concrete pavement, and
- an unbonded JPCP concrete overlay on 1.2-in. intermediate HMA layer on old concrete pavement.

Performance of these rehabilitation techniques is also compared with that of restoration (no overlay) techniques applied in 1985 on the same highway segment. The unbonded JPCP overlay showed the best performance in reflection crack elimination, structural capacity, and skid resistance through 2003. The rubblized pavement showed uniform structural capacity over time relative to the other options, based on deflection basins measured over time. The study concluded that rubblization is preferred as a rehabilitation treatment over the crack-and-seat method. The Research Division staff of the Indiana DOT will continue to evaluate the performance of these rehabilitation strategies until the year 2013.

Wisconsin (Von Quintus et al. 2007)

The Wisconsin DOT began its use of the rubblization process with a demonstration project in 1988, and has continually used this rehabilitation option for PCC pavements with extensive cracking distress. The demonstration project was a relatively small project consisting of approximately 7,000 square yards along I-43 in Walworth County. The demonstration project showed that the process was viable, and it was followed by a rubblization project along State Highway (SH) 16 in Waukesha County in 1990. After that first actual project in 1990, three projects were completed in 1992 and three projects were completed in 1993. The use of this technology has steadily increased in Wisconsin since 1995.

Prior to 1996, most of the rubblization projects included the Resonant Frequency Breaker (RFB), while after 1996 all of the projects included the use of a Multiple Head Breaker (MHB). Wisconsin also included the use of a leveling or cushion course above the rubblized PCC slabs on some of the projects completed after 1996. This leveling course consists of millings, recycled asphalt pavement (RAP), or aggregate materials. Through 2004, the Wisconsin DOT has successfully completed almost 80 projects.

Von Quintus et al. (2007) recently reviewed and analyzed the performance of 224 segments among rubblization projects in Wisconsin to determine if substantial differences exist between the performance of this rehabilitation strategy for PCC pavements used in Wisconsin and other agencies. Using the data extracted from the Wisconsin pavement management system, the performance analyses included a comparison of the condition of HMA overlays placed over intact and rubblized PCC pavements, and an extrapolation of the expected service life of the rubblized PCC pavements using empirical-mechanistic relationships.

Based on these performance analyses, the most common type of distress exhibited along a rubblized PCC pavement included transverse and longitudinal cracking, which were the result of not breaking the PCC slabs sufficiently. The use of test pits was recommended to confirm that the rubblization process was breaking the PCC into small enough pieces so that reflection cracking would not be a problem. The results of performance analyses also indicated that the rubblized PCC projects were expected to equal or exceed their design life. A minimum backcalculated elastic modulus value of 10,000 psi has been identified as the strength requirement of the foundation layers for rubblization projects.

Michigan (Baladi et al 2002; APTech 2006)

Michigan was one of the first agencies to develop a specification for PCC pavement rubblization. Several states used this specification to build their own agency-specific requirements for rubblization (Von Quintus et al. 2007). The Michigan DOT began using rubblization of concrete pavement followed by the application of a HMA overlay as a rehabilitation method for deteriorated PCC pavements in 1988 (APTech 2006). Early projects used RFB and, on one occasion, a modified whip hammer (MWH), to rubblize the pavement. However, in 1997, Michigan DOT conducted its first rubblization project using the MHB. Over the years, Michigan DOT has completed numerous rubblization projects using both the MHB and RFB equipment, with mixed performance results.

Because of concern over the performance of rubblization projects that needed rehabilitation after less than 12 years of service, Michigan DOT initiated a study in 1999 to identify the causes for underperforming rubblized concrete pavement projects. A final report on that study (Baladi, Svasdisant, and Chatti 2002) states that the prime distresses contributing to the under-performance of rubblized pavement sections are top-down cracking, joint reflective cracking, and raveling, with the majority of the top-down cracking and raveling found in areas of segregated HMA. The report also indicates that the majority of rubblized sections have segregation in the HMA due to poor construction quality. Therefore, the cause of the underperforming rubblized pavements may be more related to asphalt lay-down procedures and mixture design than the actual rubblization process itself.

Antigo Construction, Inc (Antigo) requested assistance from Applied Pavement Technology (APTech), Inc. to not only review the performance of the pavement sections rubblized using the MHB and RFB, but also to determine if a relationship exists between the categorized uniformity of rubblized pavement (as determined in previous study [Baladi et al 2002]) and the actual performance of the pavement sections (APTech 2006). The data for this particular study were obtained by extracting from the original Michigan DOT data set only those pavements rubblized using the MHB between 1997 and 2002 and those pavements rubblized using the RFB between 1988 and 2002.

The analysis of pavement sections constructed in Michigan using the MHB between 1997 and 2002 revealed that 20of the 21 MHB pavement sections evaluated had Distress Index (DI) condition ratings of good, and all of the sections had Ride Quality Index (RQI) values falling in the excellent or good category (APTech 2006). The IRI values also indicated that all MHB pavements were in good condition with values less than 95 in./mi. For those RFB sections rubblized during the same timeframe as the MHB projects (1997 to 2002), all 36 sections had DI condition ratings of good (less than 20) and ride quality according to RQI of excellent or good. Also, 65 of the 69 IRI measurements for sections rubblized since 1997 had IRI values less than the 95 in./mi threshold, further signifying that the pavement sections were providing good ride quality. For the majority of MHB and RFB pavement sections, the primary distress that occurred was longitudinal cracking, suggesting that other factors outside of the rubblization process were contributing to the observed deterioration. A qualitative comparison of the results of the uniformity of rubblization from the excavated trenches on the rubblization projects shows little correlation to the most recent Michigan DOT DI information. This report (APTech 2006) states that the more indicative cause of the few poorly performing pavement sections since 1997 seems to be due to HMA mixture problems associated with segregation and perhaps poor compaction at the edges of the pavement lane.

Alabama (Timm and Warren 2004)

The Alabama DOT (ALDOT) has completed nine rubblized sections in an 11-year period that are located on the Interstate system. The rubblized sections were constructed at different times, with different construction equipment, subbase thicknesses, HMA overlays thicknesses, and having varying amounts of traffic applied. ALDOT has implemented the rubblization process as a rehabilitation technique and collected the distress information over the years. However, these surveyed distress data were not organized to accurately quantify the performance of the rubblized pavements. ALDOT initiated a study to organize the performance data and assess the performance of the rubblized pavements (Timm and Warren 2004). Data were collected from nine rubblized sections in Alabama and a database was constructed that contained all of the surveyed distress data that pertained to the rubblized pavement sections. From this database, the rubblized pavement sections were graphically and statistically analyzed. These analyzed data led to the following conclusions. The rubblized pavement sections have performed very well and have proven to be an efficient means of rehabilitating PCC pavements. There were, however, certain patterns that were recognized in the analysis. Namely, the continuously reinforced concrete pavements exhibited the worst performance, and caution should be exercised in the future before rubblizing sections of this type.

Ohio (Rajagopal 2006)

Since 1984, the Ohio DOT has been a leader in the systematic use and evaluation of fractured slab techniques. As in other states, Ohio's use of pre-overlay fracturing began with crack-and-seat and expanded over time to include break-and-seat and rubblization. To date, 14 rubblization projects have been constructed by the Ohio DOT. On 10 of these projects, the fracturing operation was performed using the RPB; on four, the MHB was used.

A program of field evaluations (Rajagopal 2006) was undertaken on four test projects to review the condition of selected break-and-seat and rubblization projects, and also to demonstrate the ability of various pavement breakers to produce the desired breaking patterns and fractured particle sizes required by Ohio DOT specifications, as shown in Table 5. The four test projects were as follows:

- The pavement on SR-4 was rehabilitated in 1993 by breaking the underlying jointed reinforced concrete pavement with a pile hammer prior to constructing an asphalt overlay.
- The pavement on SR-36 project was rehabilitated in 1992 by rubblizing the existing jointed concrete pavement with an RPB and constructing an asphalt overlay.
- The continuous concrete pavement on I-70 was rubblized in 2005 with an MHB, in preparation for an initial asphalt overlay.
- On the I-71 project, an MHB was used to demolish the existing jointed reinforced concrete pavement and demonstrate the capabilities of MHB to produce various fracturing patterns.

Table 5. Fractured pavement particle sizes required by Ohio DOT (Rajagopal 2006)

Fracturing Technique	Particle Size	
	Predominant/Target	Maximum
Crack and Seat	4' x 4'	5'
Break and Seat	18"	30"
Rubblize and Roll	1-2"	2" (above reinforcing steel) 6" (below reinforcing steel)

At each test site, a test pit was dug and a visual assessment of the condition of the fractured pavement overlay and subbase/subgrade was made. Measurements were made of the fracturing pattern at the surface of the concrete, and gradation tests were performed to determine the particle size distribution at various depths within the fractured slab. On the MHB and RPB projects, deflection tests were performed to determine the effect of the observed breaking patterns on the stiffness of the pavement layers.

Examination of test pit material indicated that the pile hammer used in constructing the break-and-seat sections on the SR-4 project did not provide the vertical through cracking and steel debonding required by Ohio DOT specifications. Despite this, the overlay on the break-and-seat section provided vastly superior reflection crack performance than the untreated control section.

The MHB equipment used on I-70 appeared capable of providing the breaking patterns and particle sizes required by Ohio DOT specifications. However, the MHB equipment used on I-71 by a different contractor did not produce the desired results; a significant amount of large, un-cracked pieces were observed, particularly below the reinforcing steel, regardless of desired breaking pattern. On the other hand, the Resonant Pavement Breaker (RPB) equipment used on SR-36 produced fractured particle size distribution and steel debonding required by Ohio DOT specifications. The study also concluded that variation in maximum surface deflection values did not significantly change due to the breaking pattern.

This study recommended that the quality control requirements in Ohio DOT specifications be modified to require that test pits be more frequently used to ensure that the specified particle size distributions are in fact being achieved throughout the depth of the slab.

Arkansas (Rajagopal 2006)

Arkansas interstate highways are predominantly 9- to 10-in. thick JRCP. They are some of the roughest pavements in the country, being rated as the “worst roads” by *Truckers Magazine*, due to the extensive faulting at most of the transverse joints. The Arkansas State Highway and Transportation Department (AHTD) undertook the most ambitious rubblization program of 380 miles, or 60 percent of the state’s interstate highways between 2001 and 2005 (Deddens 2001 Flowers 2002; Rajagopal 2006).

At the beginning of their program, Arkansas tested and evaluated both the MHB and RPB equipment on two test projects in the year 2000 on I-40 in Brinkley and Menifee townships (Rajagopal 2006). The visual observations of the fracture patterns produced in test pit material indicated that RPB equipment generally provided smaller and more uniform particles at the surface of the rubblized layer and throughout the depth of the fractured slab. Deflection tests were conducted to compare the structural characteristics of the test pavements. The modulus values for the rubblized layer fractured with the RPB were lower (average 67 ksi) and less variable than those produced by the MHB (average 175 ksi). These results indicated that the greater stiffness of the rubblized layer produced by the MHB equipment would generally permit thinner HMA overlays; in at least some cases, the fractured layer would be too stiff to effectively prevent reflection cracking. AHTD selected the RPB as the equipment for the total program.

Colorado (LaForce 2006)

The Colorado DOT introduced rubblization technology in 1999 on a three-mile project along I-76 near Sterling, Colorado. The existing pavement on this section of I-76 was constructed in 1967 and consisted of a 2-inch emulsified, asphalt-treated base with 8 inches of JPCP. Since original construction, the pavement had received limited maintenance. In 1995, this section was overlaid with 2 inches of asphalt. In 1999, this section was rehabilitated by removing the existing 2 inches of asphalt pavement, installing edge drains, rubblizing the concrete pavement and reconditioning the shoulders, and then placing a full width 6-inch HMA pavement in three 2-inch lifts. Several rubblization projects have been undertaken since that first demonstration project in Colorado.

The Colorado DOT investigated the six-year performance of this first rubblization project (LaForce 2006). It was found that the HMA pavement has no distresses associated with reflective cracking from the old concrete pavement and has not demonstrated any settlement, permanent deformation (rutting), or other distress as a result of the rubblization process. Edge drains were shown to be effective in preventing moisture from building up under the rubblized concrete. This study recommends the usage of both the RFB and the MHB, and the edge drain in conjunction with rubblization for rubblization projects in Colorado.

Texas (Sebesta and Scullion 2006, Scullion 2006)

Rehabilitation of concrete pavements is a major issue for the Texas DOT. The department has many miles of old jointed and continuously reinforced concrete pavement, which are approaching the end of their service life. However, Texas DOT does not have a lot of experience with rubblization, and there has been little or no evaluation of the success or failure of this method (Sebesta and Scullion 2006; Scullion 2006). Scullion (2006) evaluated two rubblized sections on I-10 in Louisiana under heavy truck traffic to assess if the construction practices in use in Louisiana could be used on similar highways in Texas. Based on the FWD and GPR results, the two rubblization projects were judged to be excellent. On these projects, the rubblized concrete had a modulus value over 10 times higher than that of a traditional flexible base, and the base was draining well with no evidence of trapped water. No rutting or cracking was observed in any of the projects tested in this study. Considering the subgrade condition in this area of Louisiana where the water table is very near the surface and receives over 50 in. of rain each year, this study suggested that the main factor causing no success of rubblization in Texas is the absence of a stiff cement treated base beneath the old concrete pavement which provides good support to the rubblization process.

Federal Aviation Administration (FAA)

Within the Federal Aviation Administration (FAA) Integrated Airport System airfield infrastructure and the U.S. Department of Defense airfield inventory, there are more than 100 million square yards (83 million square meters) of PCC pavement greater than 13 inches (33) thick and more than 35 years old. These aging pavements will likely need major rehabilitation within the next 10 years (Buncher and Jones 2006). Airport agencies and the Federal Aviation Administration (FAA) have now recognized the potential of rubblization in rehabilitating old concrete airfields, given the increased use of rubblization in highways and the adoption of this practice by highway agencies (Boyer and Jones 2003).

The FAA recently published new guidelines and specifications for rubblizing airfield pavements, Engineering Brief (EB) No. 66, *Rubblized Portland Cement Concrete Base Course* (FAA 2004). These guidelines are based on industry experience and provide interim guidance. Full-scale testing at the National Airport Pavement Test Facility (NAPTF) was conducted to develop design standards for the use of this technology at airports under heavy aircraft loading (Garg and Hayhoe 2007). Three rigid airport pavements (MRC, MRG, and MRS) with 12-inch thick concrete slabs on different support systems (slab on crushed stone base, slab on grade, and slab on stabilized base) were rubblized with a RPB. All three test items were originally constructed on CBR 7 subgrade (DuPont clay). After rubblization, the rubblized concrete was rolled and paved with a 5-inch thick HMA overlay. The overlaid pavements were subjected to full-scale accelerated traffic tests under the 4-wheel landing gear configuration (with wander) and 55,000-lbs wheel load. No significant distresses were observed for 5000 passes after which the wheel load was increased to 65,000-lbs and 6-wheel landing gear was used for testing. This study concluded that the performance of MRS under a 65,000-lb wheel load suggests that rubblized concrete pavements with HMA overlay are a viable option on commercial airports, considering that MRS (slab on stabilized base) is the most representative of pavement structures that are encountered on a commercial airport in the U.S.

PROJECT DATA COLLECTION

Iowa Rubblized Pavement Projects

Rubblization Usage in Iowa

Iowa has a significant portion of PCC pavements in state highways and county roadways. Many of these pavements have deteriorated to a condition that requires rehabilitation or reconstruction. Iowa DOT recognized the potential of rubblization in rehabilitating old concrete pavements and conducted research project to rehabilitate and evaluate a severely deteriorated concrete roadway using a rubblization process (Tymkowicz and DeVrie 1995). A 3.0 km (1.9 mi.) section of L-63 in Mills county was selected and divided into 16 sections. In 1985, HMA overlay construction was done in 13 sections after rubblizing the existing pavement with a RPB and in three sections without rubblization. The variables of rubblization, drainage, and ACC overlay depths of 75 mm (3 in.), 100 mm (4 in.), and 125 mm (5 in.) were evaluated in 1995.

This research led to the following conclusions.

- The rubblization process prevents reflective cracking.
- Edge drains improved the structural rating of the rubblized roadway.
- An ACC overlay of 125 mm (5 in.) on a rubblized base provided an excellent roadway regardless of soil and drainage conditions.
- An ACC overlay of 75 mm (3 in.) on a rubblized base can provide a good roadway if

the soil structure below the rubblized base is stable and well drained.

- The Road Rater structural ratings of the rubblized test sections for this project are comparable to the non-rubblized test sections.

After this research, the use of rubblization has steadily increased in Iowa state highways and county roadways. However, there were some changes in the rubblization practices adopted in Iowa due to poor subgrade, lack of crushed aggregate base, and the use of thin concrete pavements (Jansen 2006). The modified rubblization method was proposed and adapted in the rehabilitation project of W-14 in Winneshiek county by Antigo in 2003 (see Figure 6).

Figure 6. Modified Rubblization Method proposed by Antigo (Jansen, 2006)

The main keys to the modified rubblization procedure are as follows (Jansen 2006):

- Keep the concrete pieces in place and tightly interlocked.
- Try to achieve a maximum sizing in the 12–18 in. range.
- Keep traffic off the rubblized pavement until a lift of binder is down.

Identification of Iowa Rubblization Projects

As per the original plan, the Iowa DOT's Pavement Management Information System (PMIS) database was to be used to identify the rubblization projects in this research. Most of the Iowa rubblization projects, however, belong to county roads, which are not recorded in Iowa PMIS. This led to difficulties in identifying the existing rubblized pavements. The feasible existing (in-service) rubblized pavement list, summarized in Table 6, was prepared based on discussions with the Iowa DOT districts, city, and county engineers. Among the 49 pavement sections, a total of 29 pavement sections (listed in Table 7) were for chosen the field evaluation program, with consideration to statewide location and pavement age. Core samples were taken during the field evaluation to confirm that the selected pavement sites were indeed rubblized pavements. In several instances, the construction records did not match the actual field situation, which posed a significant challenge in identifying the actual rubblized pavement sites prior to field testing. Figure 7 illustrates the different layer conditions observed underneath the HMA after coring.

Table 6. Feasible list of existing (in-service) rubblized pavement sites in Iowa

Identification No.	County	Location	Construction File No.	Project Date
1	Adair	N72	FM-C001(59)--55-01	18-Nov-03
2	Adair	G61	STP-S-C001(60)--5E-01	18-Nov-03
3	Adair	G61	FM-C001(64)--55-01	20-Jul-04
4	Allamakee	X16	STP-S-C003(30)--5E-03	27-Jan-04
5	Audubon	F24	FM-C005(31)--55-05	15-Apr-03
6	Black Hawk	Tama Road	STP-U-4082(600)—70-07	19-Apr-05
7	Black Hawk	C57	FM-07(21)--55-07	2-May-95
8	Black Hawk	D52	FM-07(23)--55-07	2-May-95
9	Black Hawk	V25	FM-07(44)--55-07	24-Mar-98
10	Black Hawk	C66	FM-07(45)--55-07	24-Mar-98
11	Black Hawk	V51	STP-S-07(46)--5E-07	24-Mar-98
12	Black Hawk	D16	FM-C007(59)--55-07	15-May-01
13	Black Hawk	V43	FM-C007(60)--55-07	15-May-01
14	Black Hawk	D38	STP-S-C007(67)--5E-07	26-Mar-02
15	Black Hawk	V43	STP-S-C007(69)--5E-07	15-Apr-03
16	Black Hawk	C57	FM-C007(81)--55-07	19-Apr-05
17	Buchanan	Multiple Routes	FM-10(17)--55-10	15-Jul-97
18	Clarke	R35	STP-S-C020(50)--5E-20	17-Feb-04
19	Clinton	Z24	FM-C023(55)--55-23	17-Feb-04
20	Davis	J40	STP-S-C026(62)--5E-26	15-Feb-05
21	Delaware	IA 3	STPN-003-8(36)--2J-28	16-Dec-03
22	Delaware	IA 3	STPN-003-8(40)--2J-28	21-Mar-06
23	Dubuque	County Road	FM-31(11)--55-31	9-Jul-96
24	Dubuque	Local Road	STP-S-31(20)--5E-31	15-Jul-97
25	Dubuque	Twelve Mile Road	STP-S-C031(32)--5E-31	27-Feb-01
26	Franklin	C23	FM-35(26)--55-35	13-Feb-98
27	Grundy	T37	FM-C038(58)--55-38	4-Jun-02
28	Grundy	Hawk Ave.	FM-C038(59)--55-38	4-Jun-02
29	Henry	Benton Ave.	FM-C044(53)--55-44	18-May-04
30	Howard	IA 9	STPN-009-7(27)--2J-45	20-Dec-05
31	Iowa	OLD U.S. 6	FM-C048(30)--55-48	3-Apr-01
32	Mils	L55	STP-S-65(40)--5E-65	21-Sep-99
33	Polk	IA 141	NHSX-141-7(22)--3H-77	24-Jul-01
34	Poweshiek	4th Ave.	STP-U-3127(5)--70-79	14-Jul-98
35	Scott	F33	STP-S-C082(29)--5E-82	17-Feb-04
36	Scott	Y68	STP-S-C082(30)--5E-82	17-Feb-04
37	Story	E18	FM-85(69)--55-85	27-Apr-99
38	Story	R50	L-FM-F198--73-85	27-Apr-99
39	Tama	U.S. 63	STPN-63-5(41)--2J-86	15-Jul-97
40	Tama	V18	FM-C086(64)--55-86	20-Jul-04
41	Taylor	J20	STP-S-C087(24)--5E-87	16-Jul-02
42	Taylor	J-55	FM-C087(30)--55-87	18-Apr-06
43	Union	H24	FM-C088(34)--55-88	21-Jun-05
44	Union	P17	FM-C088(35)--55-88	21-Jun-05
45	Van Buren	V64	STP-S-C089(42)--5E-89	17-Feb-04
46	Warren	G24	FM-91(21)--55-91	23-Sep-97
47	Winneshiek	IA 139	STP-139-0(10)--2C-96	15-May-01
48	Winneshiek	W14	FM-C096(75)--55-96	15-Jul-03
49	Woodbury	L36	FM-C097(56)--55-97	30-Apr-02

(a) **(b)**

Figure 7. Layer condition underneath HMA after coring; (a) PCC layer without rubblization (IA-139 in Winneshiek County), (b) Rubblized PCC layer (IA-3 in Delaware County)

Table 7. Selected rubblized pavement sites for field evaluation

Type of Project	I.D. No.	Project Year	Test Date	AADT 2001	AADT 2005	Layer Thickness HMA	Layer Thickness Agg.	Layer Thickness PCC
Rubblization	12	2001	7/16/07	1,160	1,280	6.6	0.0	7.5
	13	2001	7/16/07	1,710	1,340	6.4	0.0	7.9
	21	2003	7/17/07	850	740	9.7	0.0	8.7
	26	1998	7/16/07	180	120	7.5	3.0	9.2
	32	1999	11/07/07	660	820	7.1	0.0	6.1
	33-1	2001	10/22/07	17,300	18,000	7.6	0.0	9.0
	33-2	2001	10/22/07	17,300	18,000	9.2	0.0	9.8
HMA on PCC	1	2003	10/30/06	-*	-	6.3	-	6.5
	2	2003	10/30/06	-	-	7.7	-	6.0
	3	2004	10/30/06	-	-	6.0	-	13.3
	6	2005	10/25/06	-	-	9.7	-	6.0
	8	1995	10/24/06	-	-	7.1	-	6.1
	14	2002	10/25/06	-	-	10.5	-	8.3
	18	2004	10/30/06	-	-	4.1	-	7.4
	28	2002	10/24/06	-	-	5.3	-	6.1
	31	2001	10/31/06	-	-	6.8	-	9.5
	34	1998	10/31/06	-	-	5.5	-	4.5
	38	1999	10/31/06	-	-	3.8	-	6.4
	39	1997	10/24/06	-	-	5.3	-	10.3
	40	2004	10/31/06	-	-	7.5	-	6.4
	43	2005	10/30/06	-	-	7.1	-	7.1
	35	2004	7/18/07	540	445	6.2	0.0	5.9
	47	2001	7/17/07	1,050	1,010	6.0	0.0	6.8
HMA on RAC	4	2004	7/17/07	450	490	7.0	-	-
HMA on Agg.	23	1996	7/17/07	950	1,000	4.3	-	-
	24	1997	7/18/07	890	1,450	5.9	-	-
	25	2001	7/18/07	630	810	5.2	5.1	9.8
PCC on HMA	17	1997	10/25/06	-	-	PCC:1.3 HMA: 6.0	-	7.2
	46	1997	10/31/06	-	-	PCC:10.5 HMA: 0.8	-	5.8

*- = Not available.

Experimental Test Methods

The experimental test methods used to evaluate the performance of the rubblized projects included the Falling Weight Deflectometer (FWD), the Dynamic Cone Penetrometer (DCP) and visual distress surveys. Core samples were also conducted to collect in-situ material, identify the layer underneath HMA layer, and provide space for conducting the DCP test. FWD and DCP tests and coring were performed on three locations in each test section: start, middle, and end point. The visual distress survey was conducted on the entire test section. These test methods are described in detail in the following sections.

FWD

FWD has become the standard equipment for evaluating the structural condition of a pavement, due to the accuracy with which it can measure the deflected shape of a loaded pavement at appropriate rates of loading. The FWD test is conducted by applying dynamic (impulse) loads to the pavement surface, similar in magnitude and duration to that of a single, heavy, moving wheel load. The response of the pavement system is measured in terms of vertical deformation, or deflection, over a given area using seismometers (geophones). In this research, the FWD was used as the main NDT equipment to evaluate the structural condition of rubblized PCC pavement sections. Deflection data were collected using Iowa DOT's JILS-20 FWD by applying a step loading sequence of 6,000, 9,000, 12,000 and 15, 000 lbs at three different locations (start, middle, and end point) in each test project, as shown in Figure 8.

Figure 8. Picture of Iowa DOT's JILS-20 FWD equipment

DCP

DCP tests were conducted at the same locations after coring where FWD tests were conducted. The DCP tests were conducted to collect additional information about the in-situ subgrade soil properties. The DCP is an in-situ device where measurements of penetration per blow (mm/blow) are obtained. In 2003, the ASTM published a standard for use of the DCP (ASTM D6951 2003), Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications. The device works by using a standard 17.6 lb (8 kg) hammer, which is lifted to the handle and dropped to the anvil, forcing the rod to penetrate the compacted soil area. The greater the number of blows needed to penetrate the rod into the soil, the stiffer the material.

Visual Distress Survey

Visual distress surveys over the entire test section were conducted for the selected project sites identified in the field evaluation program. The distress survey methodology employed was similar to that described in the Strategic Highway Research Program's (SHRP) "Distress

Identification Manual for the Long-Term Pavement Performance (LTPP) Project" (Miller and Bellinger 2003). A distinction was made between reflective cracking and transverse cracking. Cracking was identified as "reflective cracking" when the transverse cracks were uniformly spaced (corresponding to PCC joint spacing underneath the HMA layer).

PERFORMANCE ANALYSES OF IOWA RUBBLIZED PAVEMENTS

FWD Data Analyses

Appendix A presents FWD surface deflections in each test section. FWD tests were conducted on a single location to identify the FWD sensor measurement errors. The absolute differences in deflections between two FWD tests on the same location are summarized in Table 8. No significant differences were observed, which indicated that the FWD can produce consistent results for same test materials.

Table 8. Absolute difference in FWD surface deflections between two frequency tests

Loads (Kips)	Absolute difference in FWD surface deflections (mils) between two frequency tests							
	D_0	D_8	D_{12}	D_{18}	D_{24}	D_{36}	D_{48}	D_{60}
6	0.12	0.01	0.01	0.01	0.01	0.02	0.02	0.02
9	0.03	0.00	0.02	0.02	0.01	0.00	0.01	0.04
12	0.03	0.01	0.01	0.00	0.01	0.02	0.02	0.03
15	0.23	0.02	0.01	0.02	0.00	0.01	0.06	0.34
Average	**0.10**	**0.01**	**0.01**	**0.01**	**0.01**	**0.01**	**0.03**	**0.11**

Table 9 presents averages of FWD surface deflections at the three measurement positions (start, middle, and end) and averages for different project types. The FWD surface deflections obtained for rubblized project sites are similar to those obtained for the other sites.

Table 9. Summary of FWD surface deflections results

Type of Project	I.D. No.	Temp. (°F)	Mean FWD surface deflections (mils)							
			D_0	D_8	D_{12}	D_{24}	D_{36}	D_{48}	D_{60}	D_{72}
Rubblization	12	79	12.6	10.3	9.3	7.6	6.2	4.1	2.9	2.1
	13	86	17.7	14.6	13.2	10.7	8.5	5.2	3.2	2.2
	21	67	10.2	9.1	8.6	7.7	6.8	5.3	4.1	3.2
	26	78	12.1	10.3	9.4	8.2	7.1	5.3	4.0	3.1
	32	56	12.0	11.2	10.5	9.3	8.2	6.2	4.6	3.0
	33-1	59	6.8	6.1	5.7	5.0	4.5	3.5	2.8	2.2
	33-2	63	6.5	6.0	5.7	5.1	4.6	3.6	2.9	2.3
Average		**70**	**11.1**	**9.7**	**8.9**	**7.7**	**6.5**	**4.7**	**3.5**	**2.6**
HMA on PCC	25	79	17.6	15.4	13.9	11.7	9.8	6.8	4.9	3.6
	35	82	14.0	12.7	11.6	9.9	8.4	5.8	3.9	2.7
	47	62	7.0	6.2	5.9	5.4	4.9	4.0	3.2	2.5
Average		**74**	**12.9**	**11.4**	**10.5**	**9.0**	**7.7**	**5.5**	**4.0**	**2.9**
HMA on RAC	4	74	13.6	11.2	10.0	8.3	7.0	5.0	3.7	2.7
Average		**74**	**13.6**	**11.2**	**10.0**	**8.3**	**7.0**	**5.0**	**3.7**	**2.7**
HMA on Aggre.	23	76	15.0	11.9	10.7	8.3	6.7	4.4	3.1	2.3
	24	70	16.2	14.2	13.1	11.1	9.4	6.5	4.8	3.5
Average		**73**	**15.6**	**13.1**	**11.9**	**9.7**	**8.1**	**5.4**	**3.9**	**2.9**

Rubblized Pavement Layer Moduli Backcalculation Program

Backcalculation is the ‘‘inverse’’ problem of determining material properties of pavement layers from their response to surface loading. No direct, closed-form solution is currently available to determine the layer moduli of a multilayered system given the surface and layer thicknesses. Most of the existing backcalculation programs employ iteration or optimization schemes to calculate theoretical deflections by varying the material properties until a ‘‘tolerable’’ match of measured deflection is obtained. However, in these programs, the reliability of the solution is dependent upon the seed moduli used as an input. This makes backcalculation a difficult process in which minor deviations between measured and computed deflections usually result in significantly different moduli. In many cases, various combinations of modulus values essentially produce the same deflection basin (Mehta and Roque 2003).

Recently, researchers at Iowa State University (ISU) developed user-friendly, spreadsheet-based software for layer moduli backcalculation of rubblized PCC pavements (see Figure 9). This program employs an Artificial Neural Networks (ANN)-based structural model for predicting not only the moduli of pavement layers based on FWD deflection data, but also the critical structural responses. The critical structural responses that are of interest include the horizontal tensile strain at the bottom of an HMA layer (ε_t) and the vertical compressive strain on the surface of the subgrade (ε_c). The backcalculated pavement layer moduli include the AC modulus (E_{AC}), the rubblized PCC modulus (E_{PCC}), and the subgrade modulus (E_{SG}).

Location	Pavement Thicknesses		FVD Deflections				Pavement Layer Properties				
	AC Thickness: h_{ac}	Thickness:	D-6	D-12	D-24	D-36	Strain at the AC layer: ε_{ac}	Strain at the Subgrade: ε_{sg}	AC Modulus:	PCC	Modulus:
1	14.00	10.00	6.83333	5.43137	4.24510	3.38235	-68.2	123.6	651,075	56,464	14,441
2	14.00	10.00	7.13405	5.59784	4.53372	3.62698	-59.4	129.7	532,810	95,893	13,136
3	14.00	10.00	6.75896	5.29482	4.24900	3.36454	-59.4	123.3	574,526	83,826	14,252
4	14.00	10.00	7.65101	6.30201	4.81208	3.56376	-85.6	135.5	879,909	37,501	14,636
5	14.00	10.00	7.77928	6.39933	4.93597	3.75316	-83.4	138.7	800,961	37,507	13,826
6	14.00	10.00	7.51807	6.17470	4.72892	3.51807	-83.4	133.6	871,724	37,501	14,758
7	14.00	10.00	7.47458	6.06102	4.52542	3.37627	-87.9	130.9	803,131	37,500	15,572
8	14.00	10.00	7.31391	5.87510	4.39134	3.28976	-85.8	129.2	747,356	37,501	16,008
9	14.00	10.00	7.21332	5.80821	4.37281	3.26447	-83.5	128.	783,040	37,500	16,118
10	14.00	10.00	7.59532	6.35117	5.20736	4.20401	-63.8	132.4	852,700	37,520	11,750
11	14.00	10.00	7.45431	6.19036	5.11675	4.13452	-59.6	129.9	807,918	37,706	11,807
12	14.00	10.00	7.32193	6.10865	5.02817	4.08048	-60.2	127.1	867,738	37,583	11,953
13	14.00	10.00	7.59036	6.07229	4.89923	3.88390	-66.3	137.7	560,910	57,864	12,722
14	14.00	10.00	7.58209	6.11940	4.93284	3.97015	-66.7	136.3	590,040	56,586	12,381
15	14.00	10.00	7.38416	5.90495	4.81188	3.86139	-60.9	132.8	566,389	74,471	12,543
16	14.00	10.00	7.93709	6.27815	4.87748	3.81457	-80.5	146.2	532,683	46,065	13,214
17	14.00	10.00	7.81540	6.18377	4.82781	3.76242	-78.2	144.	538,258	47,681	13,349
18	14.00	10.00	7.49803	5.86597	4.61235	3.55979	-72.7	138.9	529,394	53,448	13,928
19	14.00	10.00	6.22549	4.81373	3.96078	3.15686	-46.2	112.3	576,548	98,928	14,866
20	14.00	10.00	6.23134	4.90299	3.96269	3.22388	-52.7	112.1	660,309	103,536	14,432
21	14.00	10.00	5.86033	4.56197	3.69443	2.93902	-48.9	106.	698,734	41,391	16,710
22	14.00	10.00	7.90487	6.59071	5.22677	4.03208	-77.6	141.	850,834	37,502	12,938
23	14.00	10.00	7.74627	6.41791	5.13433	3.97015	-73.5	138.8	808,428	37,509	12,989
24	14.00	10.00	7.51393	6.20093	4.94164	3.78979	-72.7	135.	830,137	37,513	13,514
25	14.00	10.00	8.68722	7.10314	5.59978	4.40919	-84.3	157.8	577,920	37,711	11,725
26	14.00	10.00	8.73022	7.11157	5.59784	4.42132	-84.9	158.9	562,273	37,778	11,661
27	14.00	10.00	8.53007	6.87640	5.43093	4.22340	-82.3	156.9	544,252	38,147	12,222
28	14.00	10.00	9.22500	7.49659	5.43068	3.84545	-114.	157.4	635,072	37,504	13,958
29	14.00	10.00	9.12255	7.30723	5.26979	3.75319	-114.2	157.4	576,380	37,513	14,220

Figure 9. ISU rubblized PCC pavement layer moduli backcalculation program

The ANN-based structural models were developed by relating the structural responses (strains and deflections) to layer thicknesses and moduli values using the synthetic database. A synthetic database was generated using an Elastic Layer Program (ELP) by computing the critical strains for a wide range of layer thicknesses and moduli values. The HMA layer thicknesses varied from 2 in. to 12 in., and the rubblized PCC layer thicknesses varied from 6 in. to 14 in. in 2-in. increments. The moduli values ranged from 250,000 psi to 2,000,000 psi for HMA; 50,000 psi to 125,000 psi for rubblized PCC, and 5,000 psi to 50,000 psi for the subgrade. A total of 2,600 data sets were generated based on different combinations of the layer thicknesses and moduli values.

A multi-layered, feed-forward neural network trained using an error backpropagation algorithm (commonly referred to as backpropagation ANNs) was employed for the prediction of critical responses and the moduli of pavement layers. Backpropagation type ANNs are very powerful and versatile networks that can be taught mapping from one data space to another using examples of the mapping to be learned. The learning process performed by this algorithm is called backpropagation learning which is mainly an error minimization technique (Haykin 1999).

For the prediction of critical responses (ε_t and ε_c) and the moduli of pavement layers (E_{AC}, E_{PCC}, and E_{SG}), six inputs—thickness of HMA (H_1), transformed thickness of rubblized PCC layer and subbase layer (H_2), and four FWD surface deflections (D_0, D_{12}, D_{24}, and D_{36}) at 12-in.offsets starting from center deflection (D_0)—were used. The Odemark's concept of equivalent thickness was used to transform the thickness of rubblized PCC layer and subbase layer (Ceylan et al. 2005). Based on the parametric analysis, two hidden layers with 60 nodes in each layer were found to be sufficient in this case. Thus, the final ANN architecture was 6-60-60-5 (6 inputs, 60 nodes in the first and second hidden layers, and 5 output nodes, respectively).

The ANN-based strain predictions have already been validated by comparing them with the field measured strains from an instrumented trial project at highway IA-141 located in Polk County, Iowa (Ceylan and Gopalakrishnan 2007).

Layer Modulus of Rubblized Pavements

The elastic modulus for the rubblized PCC layer is an important design input required in the MEPDG. The FWD surface deflections obtained for rubblized sites were inputted into the ISU rubblized pavement layer moduli backcalculation program to predict $E_{AC,}$ $E_{PCC,}$ and E_{SG}.

The stiffness, or modulus, of HMA is very temperature-sensitive. Many researchers have reported the effect of temperature on HMA modulus. Some typical equations used in this connection are reviewed briefly below.

The 1993 AASHTO Guide (1993) adopted the following equation (1) originally developed by Witczak for use in the Asphalt Institute's Design Manual (1982):

$$\log E_{AC} = 6.451235 - 0.000164671 T^{1.92544} \qquad (1)$$

where:

E_{AC} = Modulus of asphalt concrete, psi
T = Asphalt concrete temperature, °F

The equation (1) was subsequently revised by Deacon et al.(1994) resulting in the following equation (2):

$$\log E_{AC} = 6.691635 - 0.008515T - 0.0000618T^2 \qquad (2)$$

where:

E_{AC} = Modulus of asphalt concrete, psi
T = Asphalt concrete temperature, °F

Ullidtz (1987) proposed the following equation (3) to estimate the resilient modulus of the AC with temperatures between 0 °C and 40°C:

$$E_{AC} = 15000 - 7900 \log(T) \qquad (3)$$

where:

E_{AC} = Modulus of asphalt concrete, MPa
T = Asphalt concrete temperature, °C

Ali and Lopez (1996) proposed the following equation (4) to estimate the AC modulus when the asphalt layer temperature is known at a depth of 25 mm below the surface:

$$E_{AC} = \exp(9.37196 - 0.03608145T) \tag{4}$$

where:

E_{AC} = Modulus of asphalt concrete, MPa
T = the temperature at a depth of 25 mm in the asphalt layer, ^{o}C

As can be seen in these equations, the HMA modulus is highly dependent on the HMA temperature. As a result of HMA modulus sensitivity to temperature changes, the backcalculated HMA modulus from the FWD deflections at different HMA temperature conditions must be adjusted to the modulus expected at some selected reference or characteristic temperature for the section being analyzed. A number of procedures have been developed to adjust backcalculated HMA moduli for temperature; however, most are based on limited data or for earlier deflection equipment, such as the Benkelman beam (Lukanen et al. 2000).

The semi-logarithmic format of the equation (5) relating the asphalt modulus to the mid-depth asphalt temperature was proposed by Lukanen et al. (2000) for determining the asphalt temperature adjustment factor.

$$ATAF = 10^{slope \times (T_r - T_m)} \tag{5}$$

where:

ATAF = Asphalt temperature adjustment factor
Slope = Slope of the log modulus versus temperature equation
T_r = Reference mid–depth HMA temperature
T_m = Mid-depth HMA temperature at time of measurement

Lukanen et al. (2000) recommended a slope range between -0.010 and -0.027. Since this slope range was computed from the data of 40 sites monitored in seasonal monitoring program of the Long Term Pavement Performance (LTPP) Program, the slope should be recalculated to use for local area of interest. Several local calibrated equations are available in the literature (Kim et al. 1994; Baltzer and Jansen 1994; Harichandran et al. 2001).

Noureldin (1994) reported the following equation (6) to capture the effect of temperature on HMA modulus.

$$E_{AC} = E_{AC,25} \times \frac{2747.5}{(T)^{2.46}} \tag{6}$$

where:

E_{AC} = Asphalt concrete modulus, MPa
$E_{AC,25}$ = Asphalt concrete modulus at 25°C (77°F)
T = Asphalt concrete temperature, °C

In this project, the Equation 6 was adopted to adjust the backcalculated HMA modulus for rubblized PCC pavements at different temperature conditions to the HMA modulus at a reference temperature (77°F), since the data to recalculate slope of Equation 5 in Iowa conditions is not available. Figure 10 clearly illustrates the effect of temperature on HMA modulus.

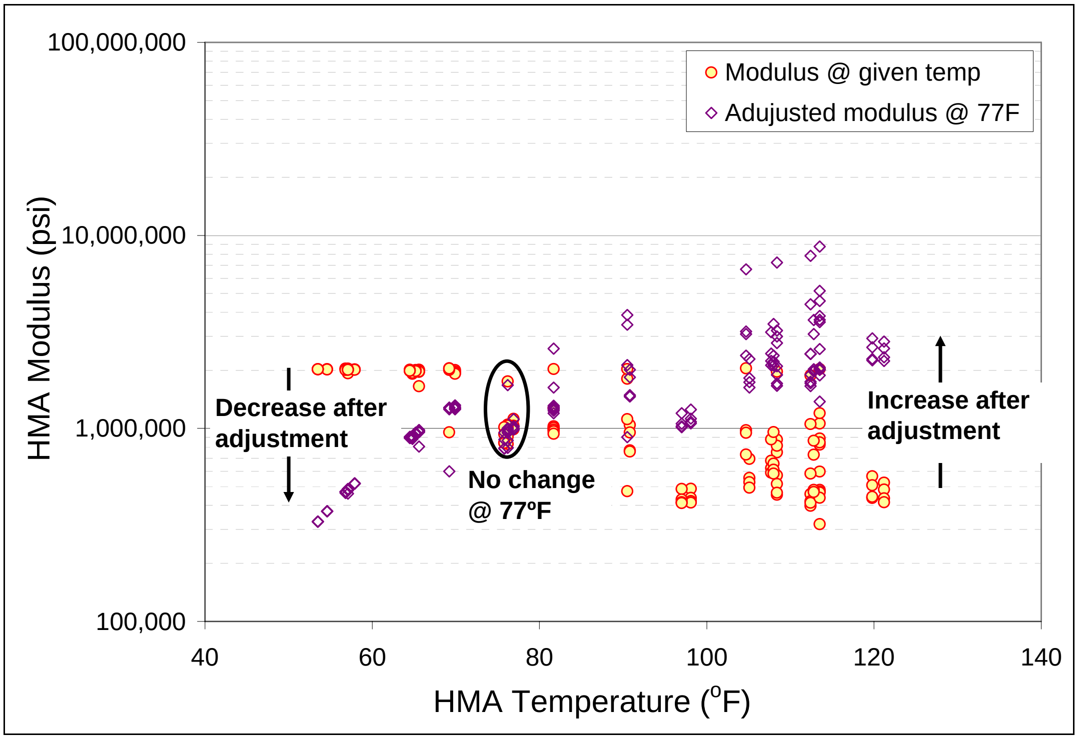

Figure 10. HMA moduli before and after adjustment to a reference temperature of 77°F.

The critical structural responses for the rubblized pavement sections at the reference temperature (77°F) were computed using the ELP. The adjusted HMA modulus (corresponding to a reference temperature of 77°F) was used in the ELP to predict the critical structural responses. Table 10 summarizes the predicted critical structural responses and layer moduli results for each of the rubblized PCC pavements.

Table 11 presents the overall statistical summary for critical structural responses and layer moduli results. The average rubblized PCC modulus in this study was found to be 78 ksi, which could be recommended as the MEPDG input for future projects in Iowa. The value of 78 ksi is close to the modulus value of 65 ksi recommended by the Wisconsin DOT study (Von Quintus et al. 2007) and lower than the default modulus value of 150 ksi currently used in MEPDG (2004). The average tensile strain value of 74 microstrain at the bottom of the HMA layer and the average vertical strain of 235 microstrain at the top of the subgrade are close to the values of 70 microstrain and 200 microstrain for long-lasting HMA pavements. The strain values for individual rubblized pavement sections, except no. 32 (L55 in Mills County), are close to the strain criteria for long-lasting HMA pavements.

Table 10. Summary of predicted critical structural responses and rubblized PCC pavement layer moduli

I.D. No.	HMA Overlay Thickness	Average				
		Strain at the bottom of the HMA layer*: ε_{AC} (microstrain)	Strain on top of the Subgrade*: ε_{SG} (microstrain)	HMA Modulus*: E_{AC} (psi)	Rubblized PCC Modulus: E_{PCC} (psi)	Subgrade Modulus: E_{SG} (psi)
12	6.6	59	181	2,771,842	70,880	19,167
13	6.4	66	217	3,180,090	38,905	13,805
21	9.7	64	210	1,146,489	75,939	10,802
26	7.5	61	205	1,918,771	99,437	13,306
32	7.1	140	483	440,423	102,999	10,671
33-1	7.6	75	216	924,723	122,079	17,738
33-2	9.2	72	168	1,057,331	47,254	17,309

* at the reference temperature (77°F)

Table 11. Overall statistical summary for predicted critical structural responses and layer moduli

	Strain at the bottom of the HMA layer*: ε_{AC} (microstrain)	Strain on top of the Subgrade*: ε_{SG} (microstrain)	HMA Modulus*: E_{AC} (psi)	Rubblized PCC Modulus: E_{PCC} (psi)	Subgrade Modulus: E_{SG} (psi)
Average	74	235	1,634,239	78,131	14,539
S.D.	32	107	1,021,770	44,924	4,082
Max	198	577	3,180,090	162,482	20,309
Min	26	97	440,423	37,500	9,566

* at the reference temperature (77°F)

Layer Coefficient of Rubblized PCC

Many DOTs and counties use the 1993 AASHTO HMA Overlay Thickness Design method to calculate HMA overlay thickness. The rubblized PCC structural layer coefficient (a_2) is required in this design method. The following is an empirical equation (7), relating a_2 to rubblized PCC layer modulus, suggested by Galal et al. (1999) to provide results equivalent to the layer coefficients derived at the AASHO Road Test.

$$a_2 = 0.0045\sqrt[3]{E_2} \tag{7}$$

where:

a_2 = rubblized PCC layer coefficient
E_2 = rubblized PCC layer modulus

For this study, the rubblized PCC layer coefficient values obtained for individual rubblized PCC sections and the overall statistical summary are provided in Tables 12 and 13, respectively. The average PCC layer coefficient value in this study was found to be 0.19. This is consistent with the range (0.14 to 0.20) used by Arkansas, Michigan, Mississippi, Ohio, and Pennsylvania (Von Quintus et al. 2007). However, those values are lower than a value of around 0.25 used by Minnesota, New York, Indiana and Wisconsin (Galal et al. 1999; Von Quintus et al. 2007).

Table 12. Rubblized PCC layer coefficient (a_2) for individual pavements

I.D. No.	HMA overlay thickness	Average rubblized PCC layer coefficient (a_2)
12	6.6	0.19
13	6.4	0.15
21	9.7	0.19
26	7.5	0.21
32	7.1	0.20
33-1	7.6	0.22
33-2	9.2	0.16

Table 13. Overall statistical summary for rubblized PCC layer coefficient (a_2)

	Rubblized PCC layer coefficient (a_2)
Average	0.19
S.D.	0.04
Max.	0.25
Min.	0.15

DCP and Visual Distress Survey Results

DCP tests and visual distress surveys were performed on the 29 pavement sections listed in Table 7. The test activities were more concentrated on the test sections identified as rubblized pavements, determined after coring. Some of the data were not available from the non-rubblized test sections.

To represent DCP measures at different depths in each location, the average rate of penetration or penetration index ($DCPI_{wtag}$) is determined by calculating the weighted average using the following equation (8) (Sawangsuriya and Edil 2004):

$$DCPI_{wtag} = \frac{1}{H}\sum_{i}^{N}\left[(DCPI)_i \times (z)_i\right] \quad (8)$$

where:

H = total penetration depth
z = layer thickness
DCPI = penetration index for z, mm/blow

The rate of penetration (DCPI) has been correlated to the California Bearing Ratio (CBR, percent), an in-situ strength parameter (ASTM D6951 2003). The DCPI-CBR correlation for soils other than CL below CBR 10% and CH soils is as follows (9):

$$CBR = \frac{292}{DCPI^{1.12}} \tag{9}$$

The CBR has been correlated to the resilient modulus (M_r, psi), an input parameter representing soil material strength in MEPDG (NCHRP 2004). The M_r-CBR correlation is as follows (10):

$$M_r = 2555(CBR)^{0.64} \tag{10}$$

Appendix B provides plots of DCP data collected in this study. Table 14 presents the summary of DCP test results. The average subgrade modulus value for rubblized sections is lower than that of HMA-over-aggregate base sections. However, the average subgrade modulus value for rubblized sections is close to that found in other structural sections. The average rubblized pavement subgrade modulus value of 12 ksi meets the minimum strength requirement (10 ksi) of the foundation layers for rubblization project specified by the Wisconsin DOT (2007). Considering that the DCP tests were conducted in summer, the results seem to indicate that the foundation layer of Iowa rubblized sections can provide sufficient strength. Table 15 presents the comparison of M_r values obtained from DCP with those obtained using FWD. The overall average backcalculated M_r value of 14 ksi obtained from FWD data using the ISU ANN-based backcalculation program is slightly higher than the M_r value of 12 ksi obtained from DCP test results.

Table 14. Summary of DCP test results

Type of Project	I.D. No.	DCP Test Results					
		Average for 3 measurement locations			S.D. for 3 measurement locations		
		$DCPI_{wtag}$ (mm/blow)	CBR (%)	Mr (psi)	$DCPI_{wtag}$ (mm/blow)	CBR (%)	Mr (psi)
Rubblization	12	17.7	23.5	17,485.4	10.6	24.3	11,920.0
	13	25.4	10.5	11,109.2	13.5	5.5	3,885.8
	21	30.0	7.5	8,994.9	12.2	2.4	1,899.2
	26	35.3	4.9	6,533.8	12.4	1.0	1,388.0
	32	24.3	9.6	10,642.8	2.5	1.6	1,079.2
	33-1	17.3	48.1	26,114.6	19.2	58.9	25,117.9
	33-2	33.2	7.4	8,924.3	17.0	4.0	3,240.0
Average		**26.2**	**15.9**	**12,829.3**	**12.5**	**14.0**	**6,932.9**
HMA on PCC	1	39.0	5.6	7,547.6	6.6	1.1	970.6
	2	32.7	8.5	9,592.1	17.4	4.3	3,312.8
	3	40.0	4.8	6,916.8	n/a	n/a	n/a
	6	12.1	24.9	18,815.7	0.1	5.6	1,802.4
	8	24.2	10.4	11,049.7	6.5	3.7	2,394.2
	14	24.6	9.6	10,667.0	6.2	2.5	1,795.9
	18	48.9	4.9	6,807.9	11.5	1.8	1,469.6
	28	29.9	8.1	9,442.5	6.0	1.2	855.4
	31	31.5	6.5	8,377.5	2.3	0.9	662.5
	34	29.7	15.2	12,950.7	19.1	17.0	9,677.3
	38	31.9	8.3	9,489.1	14.8	4.1	3,113.4
	39	31.2	8.5	9,747.9	19.0	3.8	3,146.4
	40	39.5	6.3	8,049.5	7.9	2.4	1,853.7
	43	28.3	7.9	9,436.7	3.0	1.1	762.9
	35	23.8	12.9	12,519.9	14.2	7.2	4,657.0
	47	22.2	10.1	11,104.0	4.9	2.3	1,615.8
Average		**30.6**	**9.5**	**10,157.2**	**9.3**	**3.9**	**2,539.3**
HMA on RAC	4	30.8	16.6	13,630.1	26.3	16.9	9,675.7
Average		**30.8**	**16.6**	**13,630.1**	**26.3**	**16.9**	**9,675.7**
HMA on Agg.	23	2.8	114.5	51,613.6	-*	-	-
	24	8.9	45.4	27,584.6	-	-	-
	25	44.9	5.5	7,323.7	16.7	2.5	2,218.3
Average		**18.9**	**55.1**	**28,840.6**	**16.7**	**2.5**	**2,218.3**
PCC on HMA	17	20.2	12.8	12,599.7	4.0	4.1	2,355.6
	46	36.2	5.4	7,484.1	-	-	-
Average		**28.2**	**9.1**	**10,041.9**	**4.0**	**4.1**	**2,355.6**

*- = Not available.

Table 15. Comparison of M_r values from DCP and FWD

I.D. No.	Mean of M_r (psi)		S.D of M_r (psi)	
	DCP	FWD	DCP	FWD
12	17,485	19,167	11,920	1,468
13	11,109	13,805	3,886	3,045
21	8,995	10,802	1,899	881
26	6,534	13,306	1,388	4,799
32	10,643	10,671	1,079	840
33-1	26,115	17,738	25,118	4,050
33-2	8,924	17,309	3,240	1,475
Total	**12,829**	**14,539**	**6,933**	**4,082**

Appendix C provides selected pictures from the visual distress surveys. Visual distress survey results are also summarized in Table 16. In general, no load-associated distresses, such as fatigue cracking, were found in any of the test sections as shown in Figure 11. The predominant distresses observed in the rubblized PCC sections were longitudinal cracking and low-temperature cracking, as shown in Figures 12 and 13, respectively. No reflection cracking was observed in these rubblized PCC sections. However, some of the HMA-over-PCC sections without rubblization, especially IA-139 in Winneshiek County (I.D. No. 47), showed high-severity reflection cracking, as shown in Figure 14.

Table 16. Summary of visual distress survey results

Type of Project	I.D. No.	Project Year	AADT		Layer Thickness			Visual Distress Survey Results
			2001	2005	HMA	Agg.	PCC	
Rubblization	12	2001	1,160	1,280	6.6	0.0	7.5	11 low temperature cracks
	13	2001	1,710	1,340	6.4	0.0	7.9	1 Block and 8 low temperature cracks
	21	2003	850	740	9.7	0.0	8.7	2 longitudinal cracking on wheel paths (about 3 mile) and 9 low temperature cracks
	26	1998	180	120	7.5	3.0	9.2	No cracks
	32	1999	660	820	7.1	0.0	6.1	14 low temperature cracks
	33-1	2001	17,300	18,000	7.6	0.0	9.0	14 Longitudinal cracks, 3 low temperature cracks
	33-2	2001	17,300	18,000	9.2	0.0	9.8	2 Longitudinal cracks
HMA on PCC	1	2003	-*	-	6.3	-	6.5	Pavement in good condition.
	2	2003	-	-	7.7	-	6.0	Pavement in good condition.
	3	2004	-	-	6.0	-	13.3	-
	6	2005	-	-	9.7	-	6.0	Pavement in good condition.
	8	1995	-	-	7.1	-	6.1	Reflective crack at start position
	14	2002	-	-	10.5	-	8.3	Pavement in good condition.
	18	2004	-	-	4.1	-	7.4	Pavement in good condition
	28	2002	-	-	5.3	-	6.1	Pavement in good condition
	31	2001	-	-	6.8	-	9.5	500 ft after a high severity reflective crack
	34	1998	-	-	5.5	-	4.5	Pavement good condition.
	38	1999	-	-	3.8	-	6.4	5-6 sealed low severity low temperature cracks
	39	1997	-	-	5.3	-	10.3	Reflective cracks at start and end position
	40	2004	-	-	7.5	-	6.4	Pavement in good condition.
	43	2005	-	-	7.1	-	7.1	Pavement in good condition.
	35	2004	540	445	6.2	0.0	5.9	24 low temperature cracks
	47	2001	1,050	1,010	6.0	0.0	6.8	More than 10,000 reflection cracks
HMA on RAC	4	2004	450	490	7.0	-	-	-
HMA on Agg.	23	1996	950	1,000	4.3	-	-	-
	24	1997	890	1,450	5.9	-	-	-
	25	2001	630	810	5.2	5.1	9.8	6 low temperature cracks
PCC on HMA	17	1997	-	-	PCC:1.3 HMA: 6.0	-	7.2	PCC overlay in good condition
	46	1997	-	-	PCC:10.5 HMA: 0.8	-	5.8	PCC overlay in good condition

*- = Not available.

Figure 11. Picture of distress-free HMA surface on rubblized PCC (I.D. No. 12: C23 in Franklin County)

Figure 12. Picture of longitudinal cracking on HMA-overlaid rubblized PCC (I.D. No. 21: IA3 in Delaware County)

Figure 13. Picture of low-temperature cracking on HMA-overlaid rubblized PCC (I.D. No. 12: D16 in Blackhawk County)

Figure 14. Picture of reflection cracking on HMA-overlaid PCC pavement (I.D. No. 47: IA139 in Winneshiek County)

Validation of the M-E Design Procedure Developed during Phase I Study

During the first phase of this study (IHRB TR-473), a mechanistic-empirical (M-E) design approach for the HMA overlay thickness design for fractured PCC pavements was proposed (Ceylan et al. 2005). In this design procedure, failure criteria, such as the tensile strain at the bottom of HMA layer and the vertical compressive strain on the surface of subgrade, are used to consider HMA fatigue and subgrade rutting, respectively. The developed M-E design software system was also implemented in a Visual Basic computer program with a user-friendly interface (see Figure 15). One of the main objectives in this Phase II study was to validate the M-E HMA overlay thickness design procedure developed during the Phase I study. This task was achieved by comparing the actual HMA overlay thicknesses with the thicknesses predicted by the design software for the rubblized PCC sections selected in this study. The comparisons are summarized in Table 17.

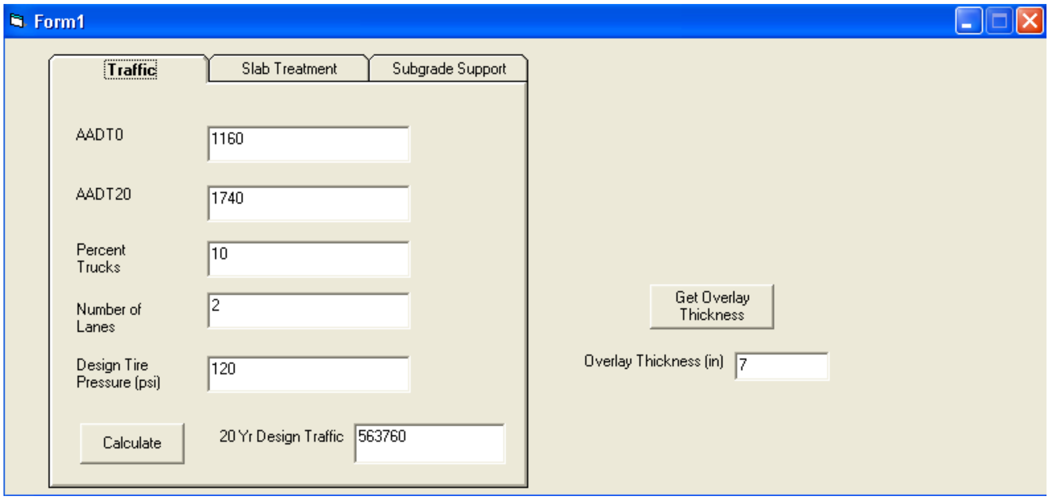

Figure 15. Screenshot of HMA overlay thickness design Visual Basic Program

Table 17. Comparison between actual and computed HMA overlay thickness

I.D. No.	HMA Overlay Thickness (in.)	
	Actual	Calculated
12	6.6	7.0
13	6.4	8.0
21	9.7	7.5
26	7.5	6.5
32	7.1	7.5
33-1	7.6	11.0
33-2	9.2	12.0
Average	7.7	8.5
S.D	1.3	2.1

The actual HMA overlay thickness of in-service rubblized PCC sections are in agreement with the results obtained from the developed HMA overlay thickness design software. No load-associated distresses, such as fatigue cracking and rutting, were observed in the rubblized PCC sections evaluated, indicating that the design method developed in Phase I can estimate the HMA overlay thickness reasonably well to achieve long-lasting performance of HMA overlay on rubblized PCC pavements.

SUMMARY

Findings and Conclusions

The structural condition of existing rubblized concrete pavements across Iowa was evaluated through Falling Weight Deflectometer (FWD) tests, Dynamic Cone Penetrometer (DCP) tests, visual pavement distress surveys, etc. Through backcalculation of FWD deflection data using the ISU layer moduli backcalculation program, the rubblized layer moduli values were determined for various projects and compared with each other for correlating with the long-term pavement performance. The AASHTO structural layer coefficient for the rubblized pavement layer was also calculated using the rubblized layer modulus values. The M-E design procedure developed during the Phase I study was validated by comparing the actual and the computed HMA overlay thicknesses. Based on the results of this study and other agency studies documented in the literature, the followings findings and conclusions are drawn:

- Rubblization is a valid option to use in the rehabilitation of PCC provided the foundation is strong enough to support construction operations during the Rubblization process.
- The M-E HMA overlay thickness design software developed during the first phase of this study seems to estimate the HMA overlay thickness reasonably well to achieve long-lasting performance of HMA overlay pavements with rubblization.
- Iowa's rubblized pavement sections are performing very well. The predominant distresses exhibited on HMA-overlaid rubblized PCC sections are non-load associated distresses, such as low-temperature cracking and/or longitudinal cracking.
- The average rubblized PCC modulus of the rubblized layer in this study was found to be 78 ksi, which is close to the modulus value of 65 ksi recommended by the Wisconsin DOT study.
- The average rubblized PCC layer coefficient value in this study was found to be 0.19, which is consistent with that used by Arkansas, Michigan, Mississippi, Ohio, and Pennsylvania.
- The average tensile strain value of 74 $\mu\varepsilon$ (microstrain) at the bottom of HMA layer and the average vertical strain of 235 $\mu\varepsilon$ on top of the subgrade are close to the values of 70 $\mu\varepsilon$ and 200 $\mu\varepsilon$ recommended for long-lasting HMA pavements.
- The ISU ANN-based backcalculation program provides good predictions for subgrade modulus.

Recommendations

The following recommendations are made to suggest activities that the Iowa DOT could consider to confirm the design criteria and decision factors for use of rubblization in Iowa.

- Iowa DOT should continue to use PCC rubblization as a valid pavement rehabilitation strategy
- Iowa DOT should confirm the minimum foundation support condition or elastic modulus of the foundation. Wisconsin DOT specifies this value as 10 ksi.

- A structural layer coefficient of 0.19 is recommended for use in AASHTO design method and a layer modulus value of 78 ksi is recommended for use in MEPDG design method.

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APPENDIX A: FWD TEST RESULTS

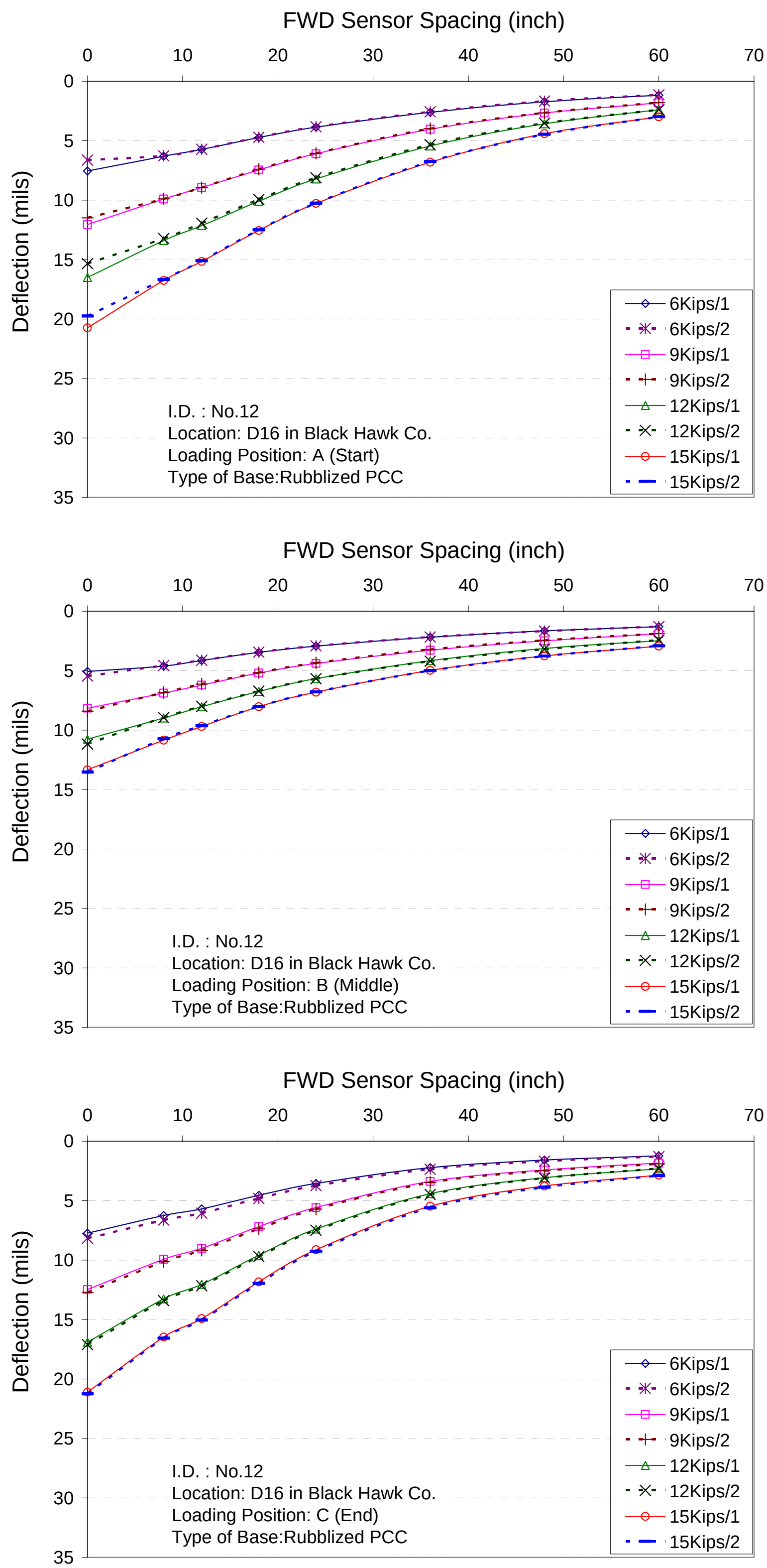


Figure A.1. D16 in Black Hawk County, IA (I.D. No. 12): Rubblization

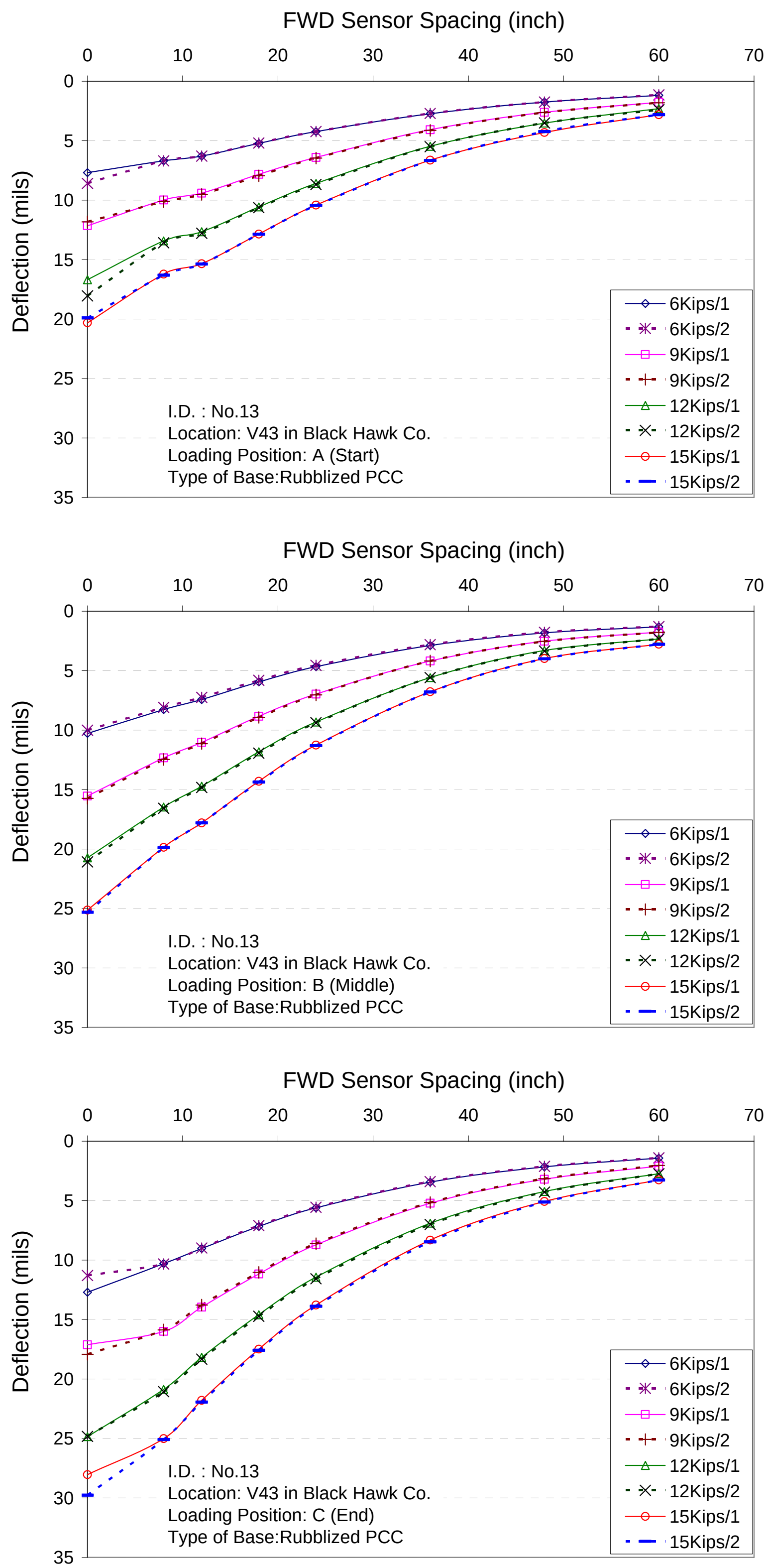
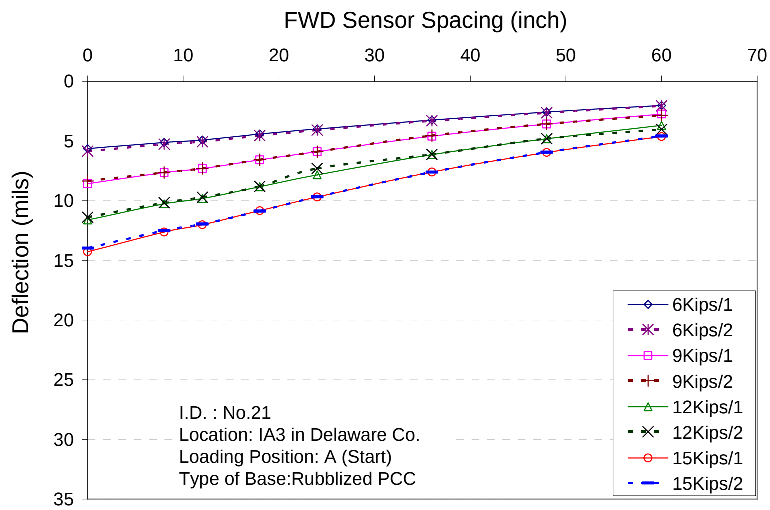
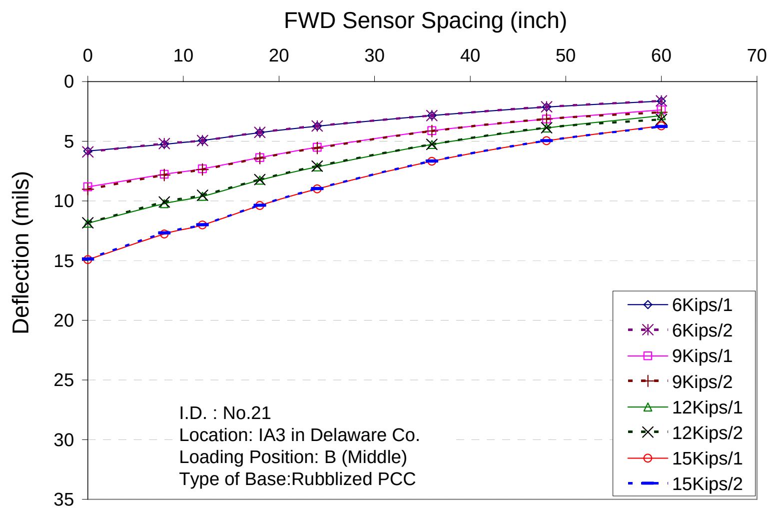


Figure A.2. V43 in Black Hawk County, IA (I.D. No. 13): Rubblization





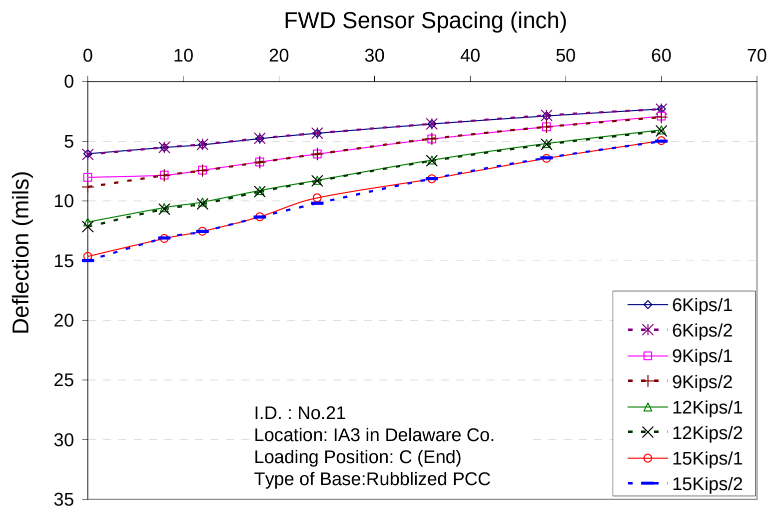
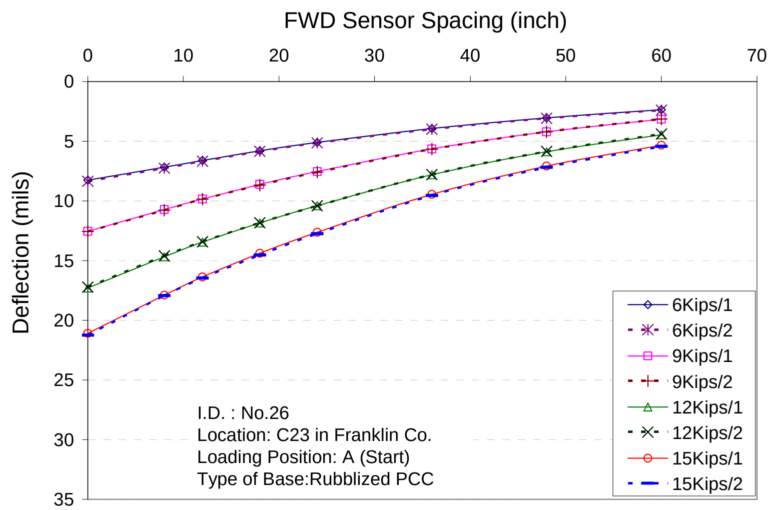
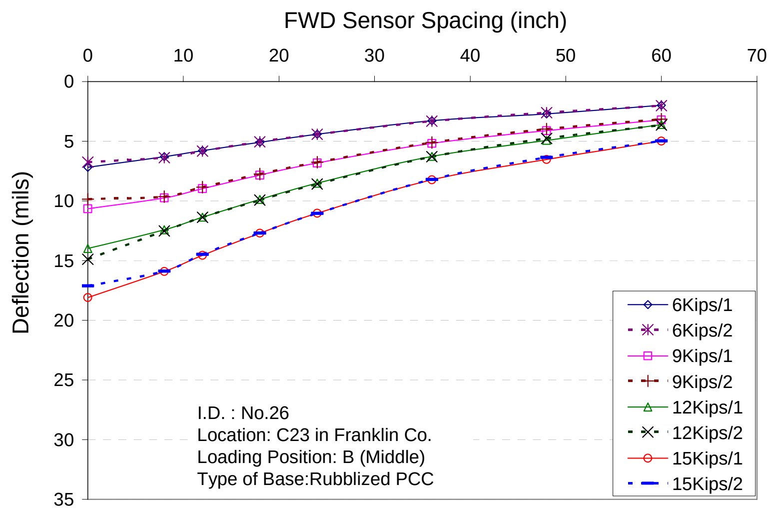


Figure A.3. IA3 in Delaware County, IA (I.D. No. 21): Rubblization





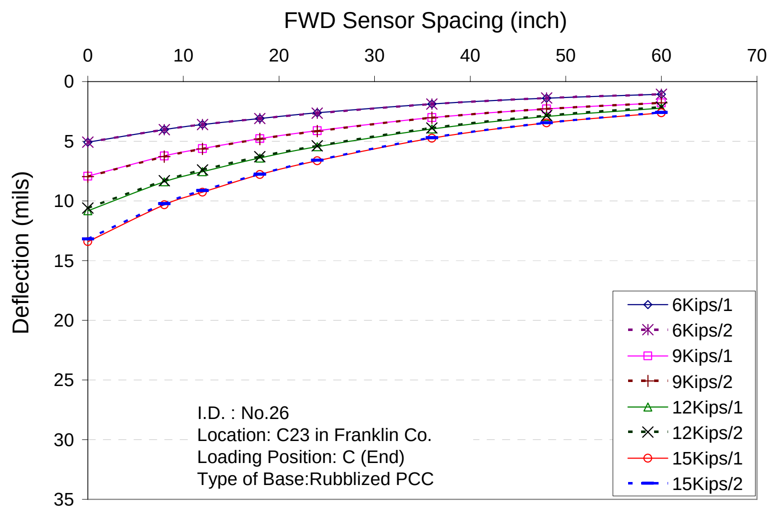


Figure A.4. C23 in Franklin County, IA (I.D. No. 26): Rubblization

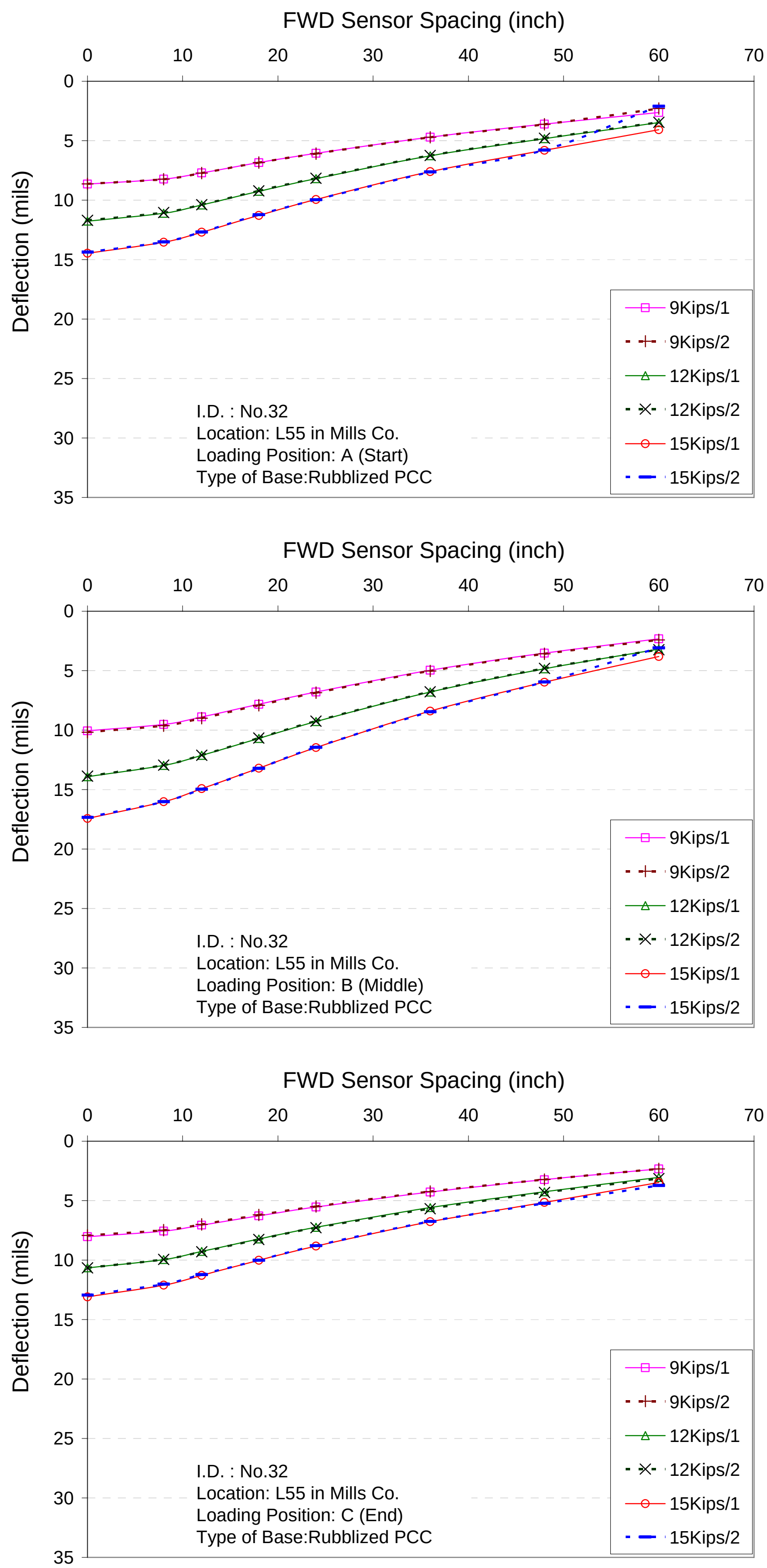
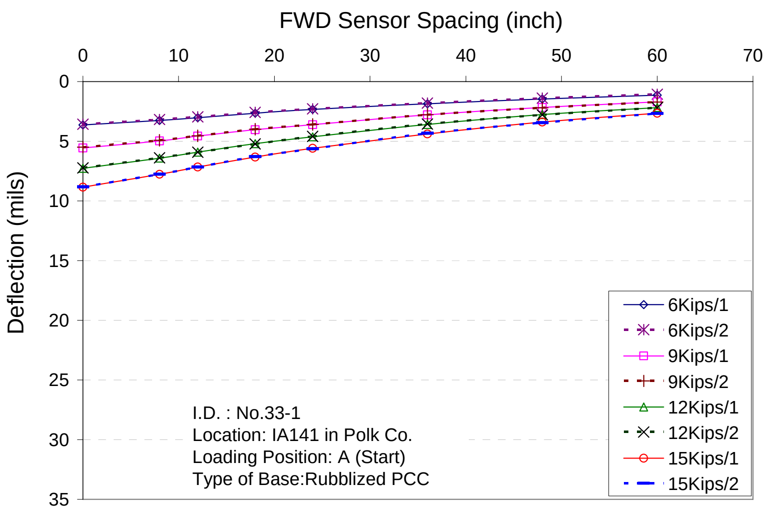


Figure A.5. L55 in Mills County, IA (I.D. No. 32): Rubblization



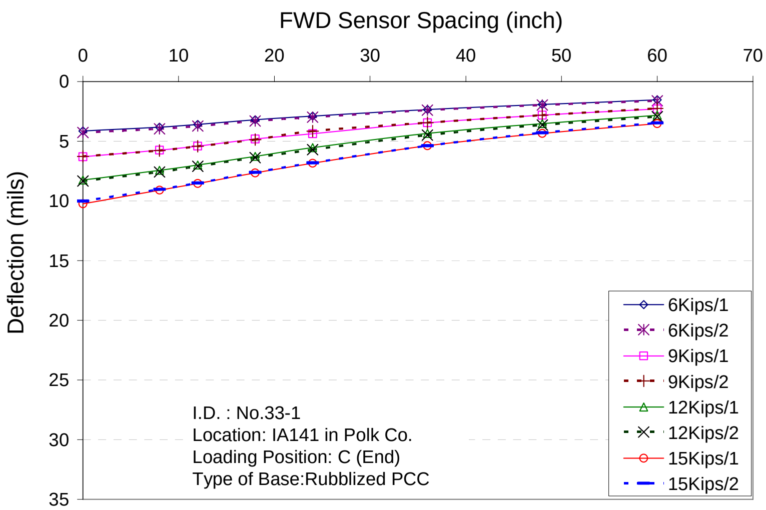
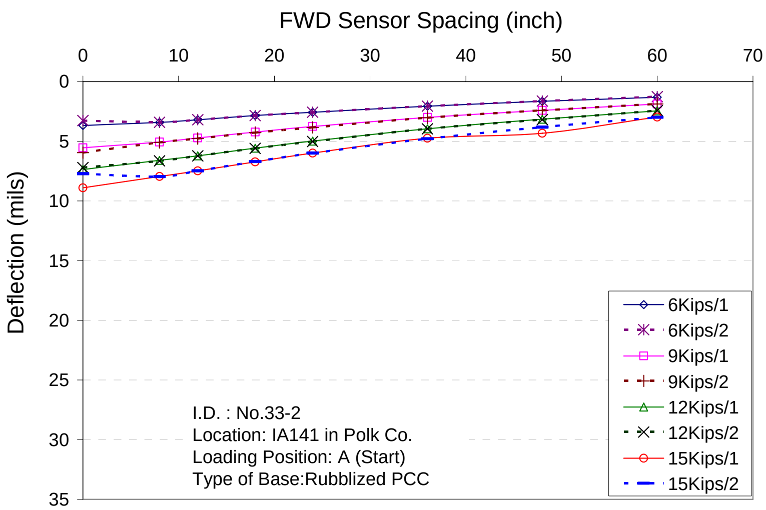


Figure A.6. IA141 in Polk County, IA (I.D. No. 33-1): Rubblization



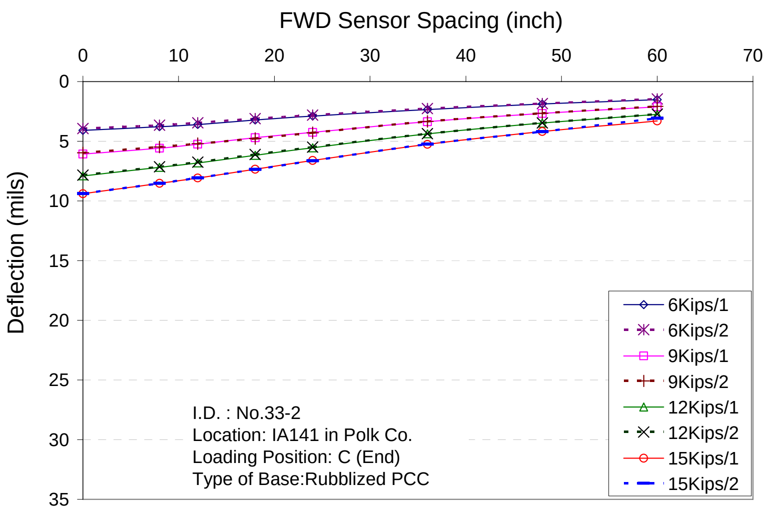


Figure A.7. IA141 in Polk County, IA (I.D. No. 33-2): Rubblization

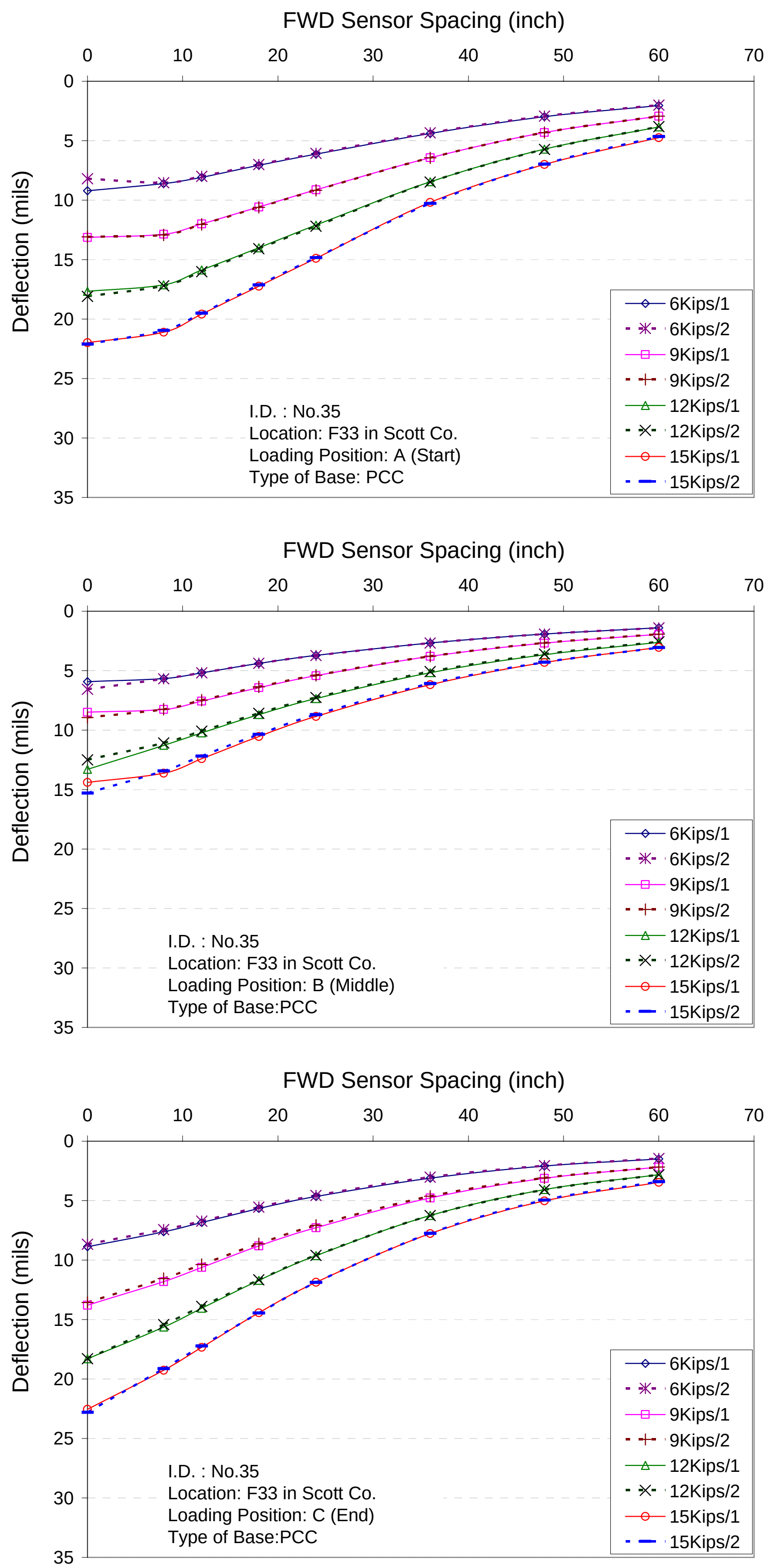


Figure A.8. F33 in Scott County, IA (I.D. No. 35): HMA on PCC

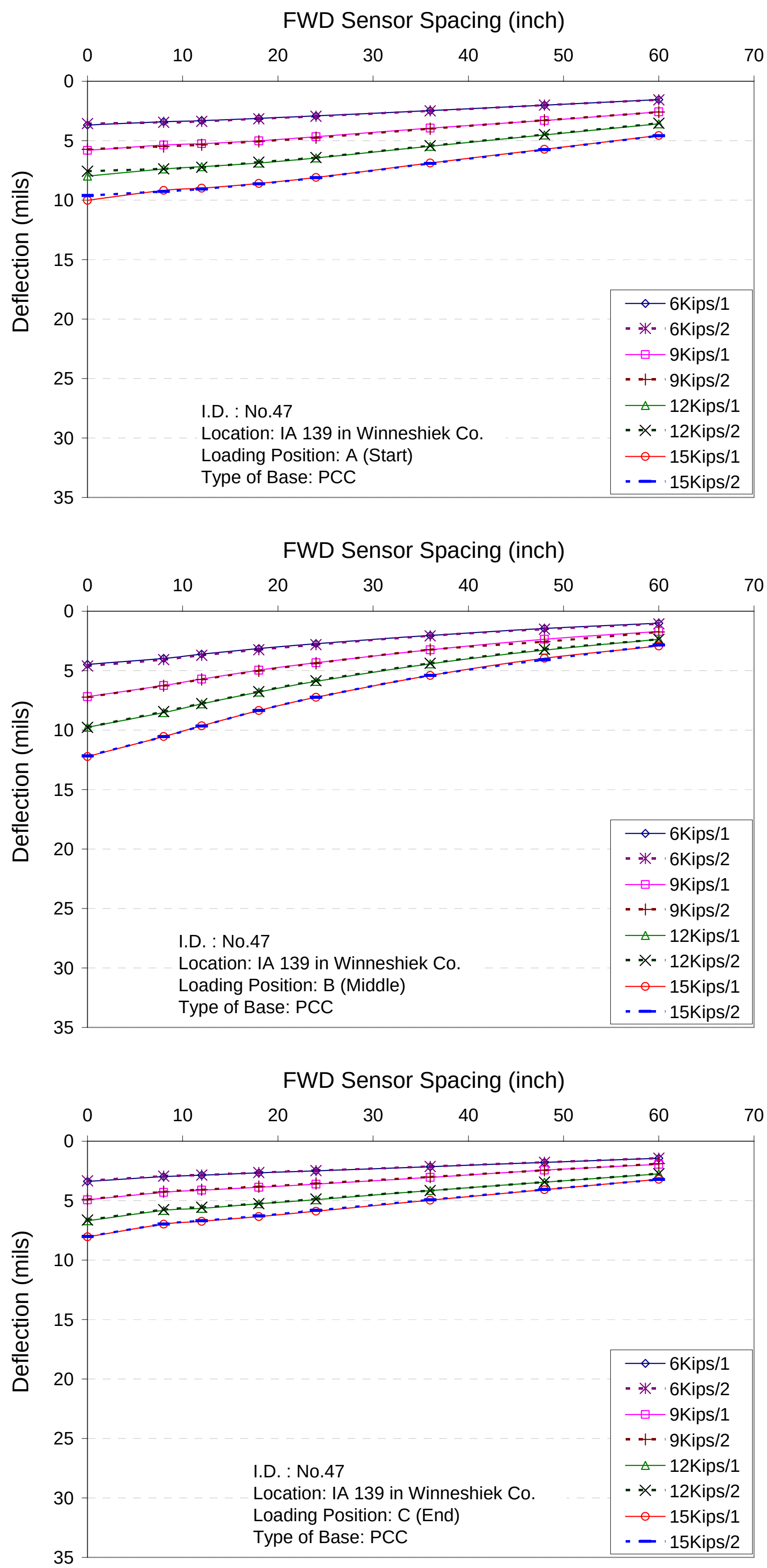


Figure A.9. IA139 in Winneshiek County, IA (I.D. No. 47): HMA on PCC

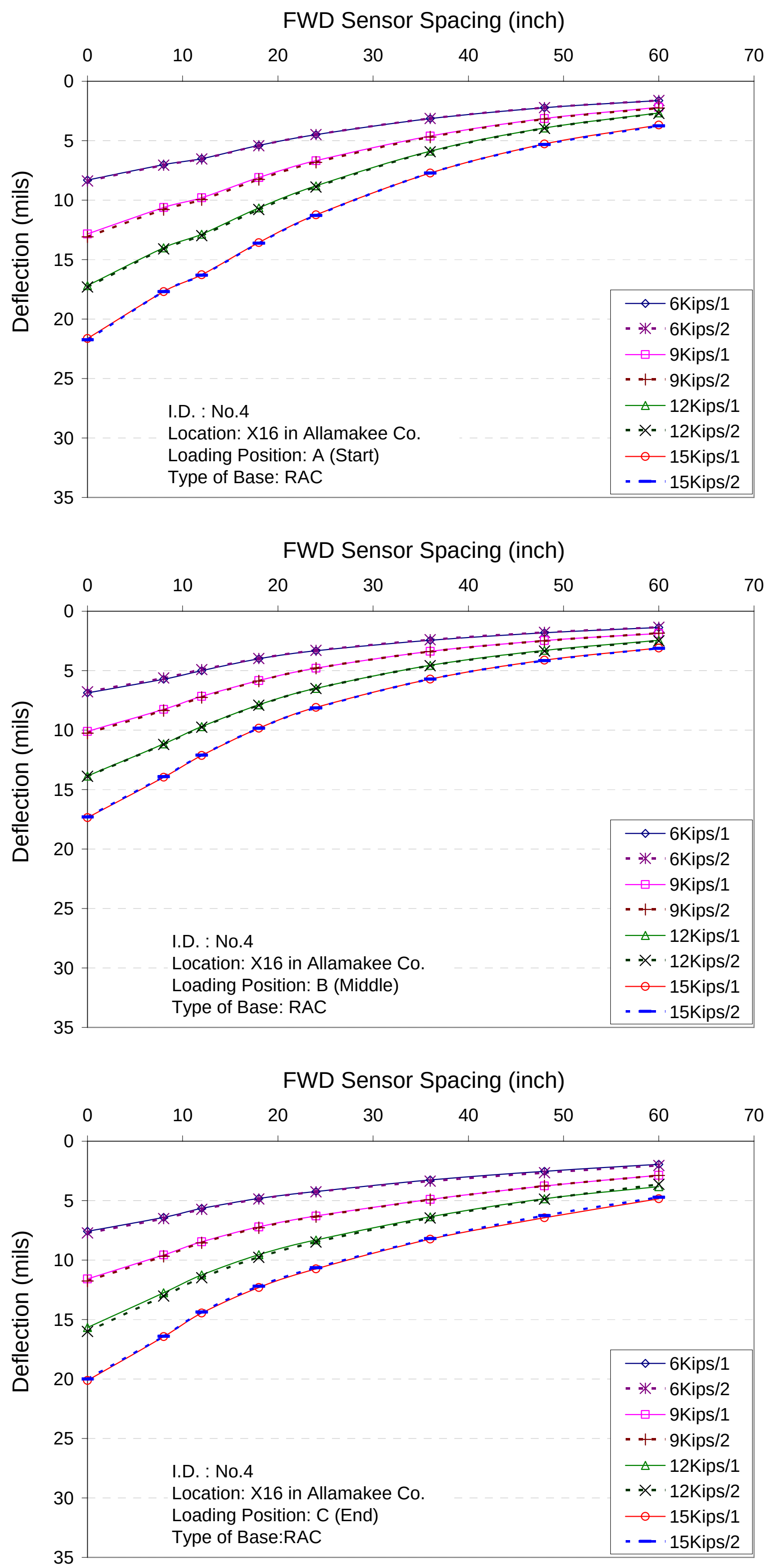
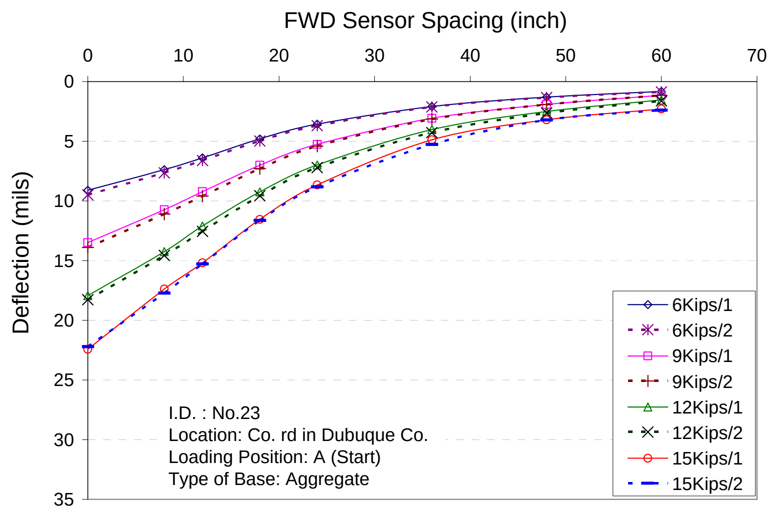


Figure A.10. X16 in Allamakee County, IA (I.D. No. 4): HMA on RAC



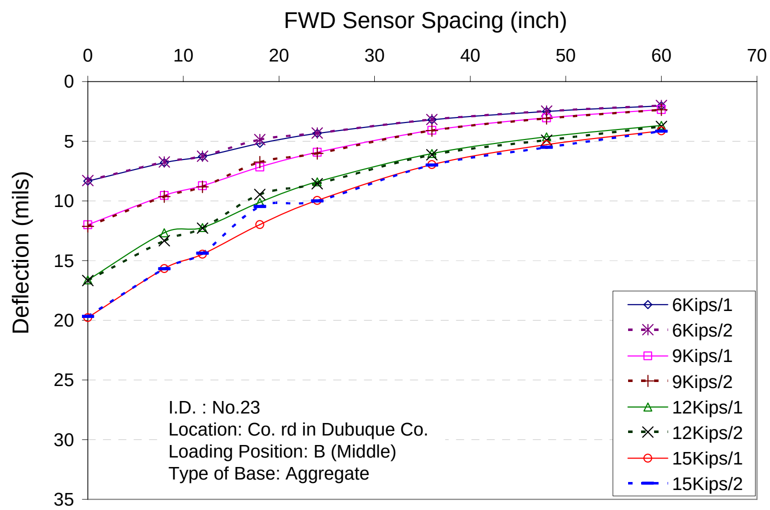


Figure A.11. County Road in Dubuque County, IA (I.D. No. 23): HMA on Aggregate

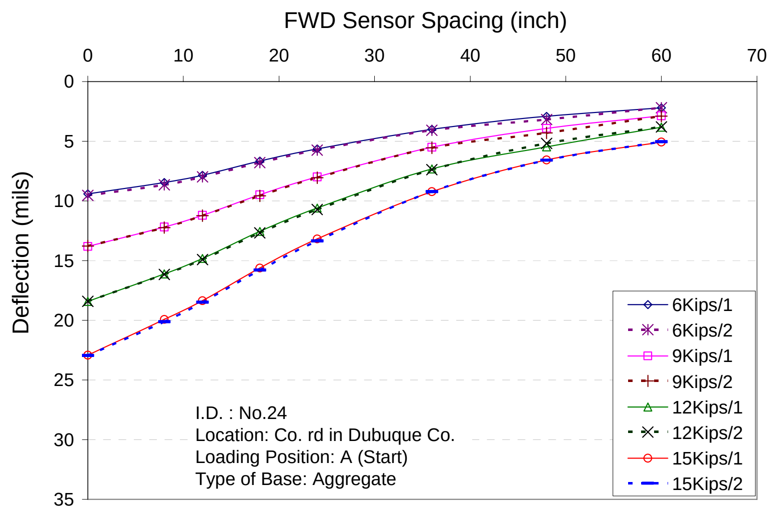
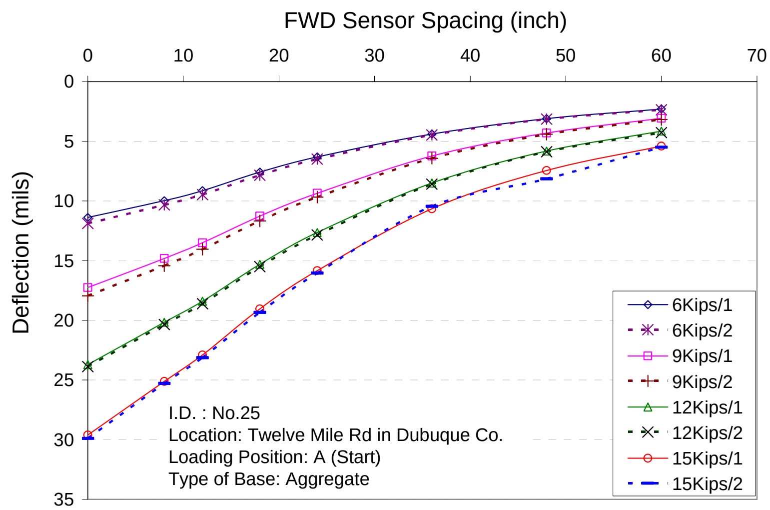
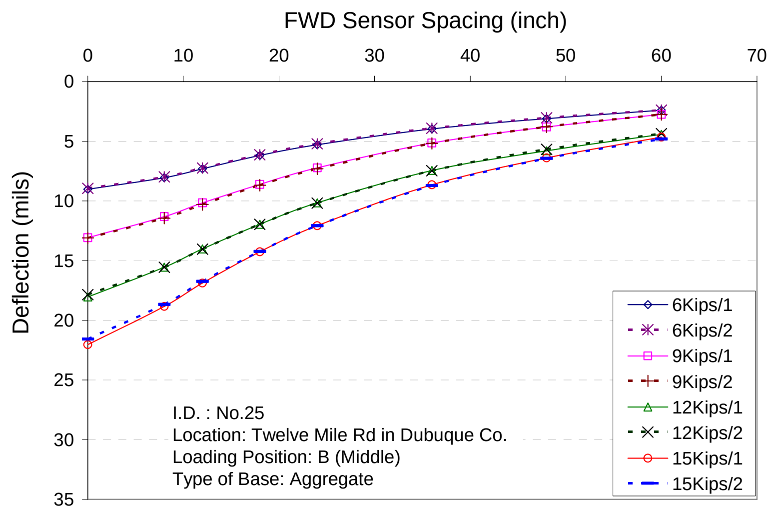


Figure A.12. County Road in Dubuque County, IA (I.D. No. 24): HMA on Aggregate





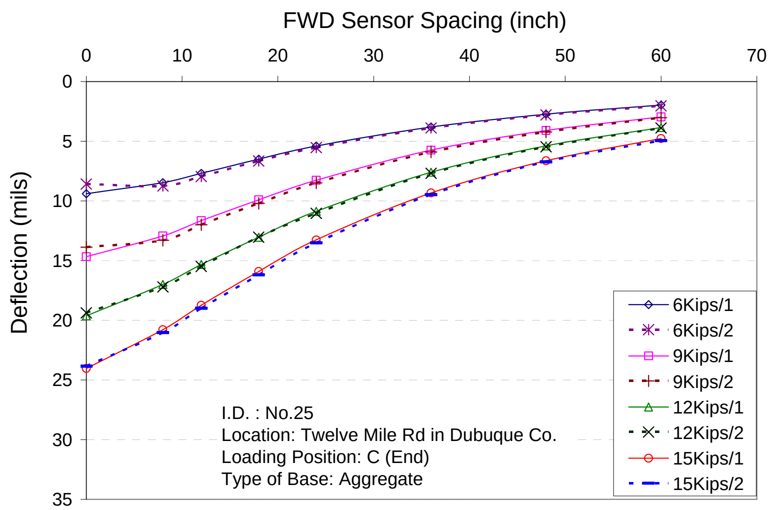
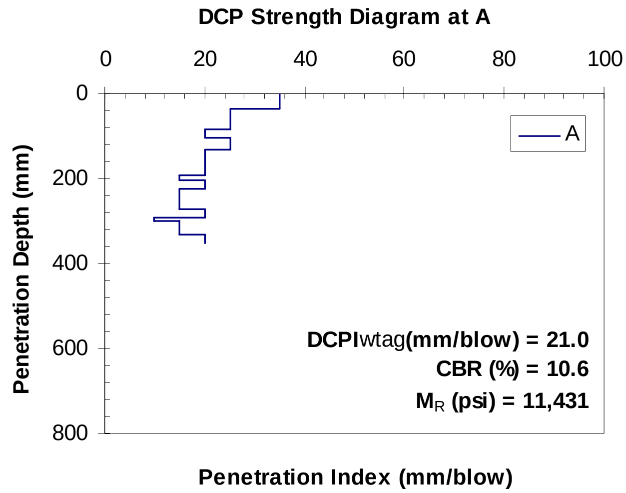
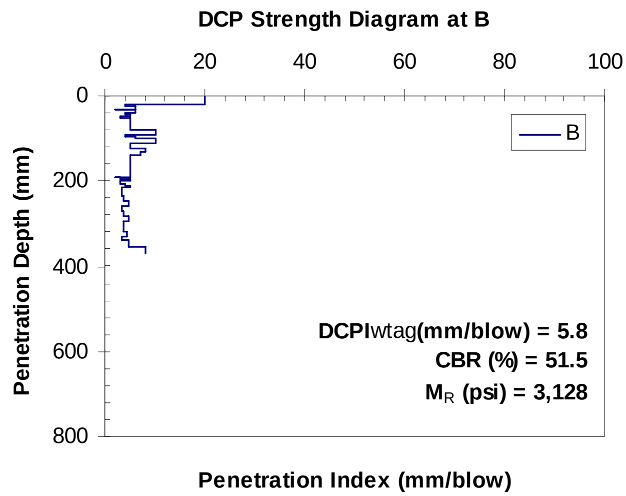


Figure A.13. Twelve Mile Road in Dubuque County, IA (I.D. No. 25): HMA on Aggregate

APPENDIX B: DCP TEST RESULTS





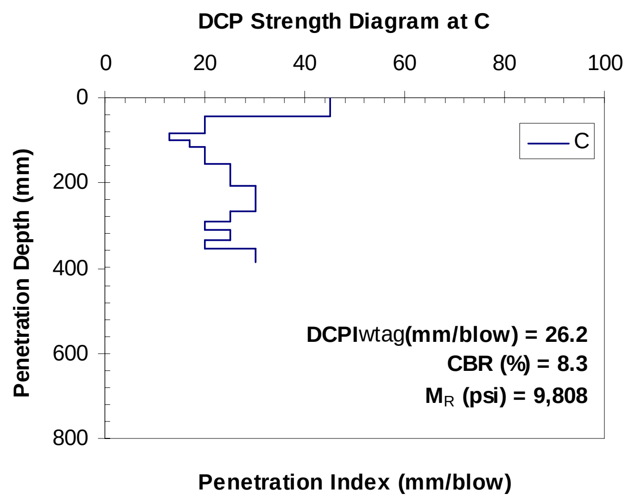
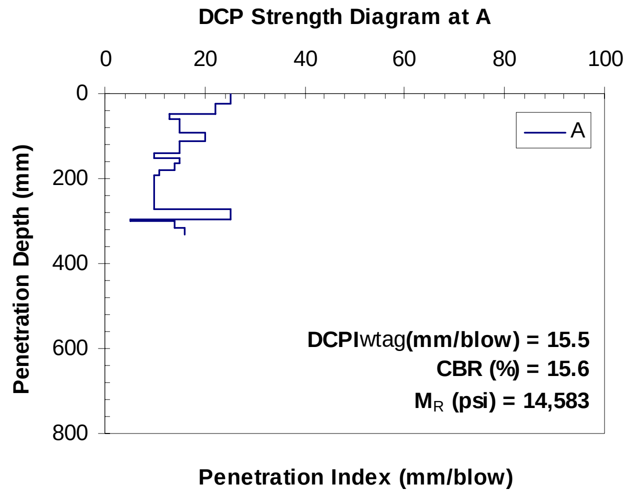
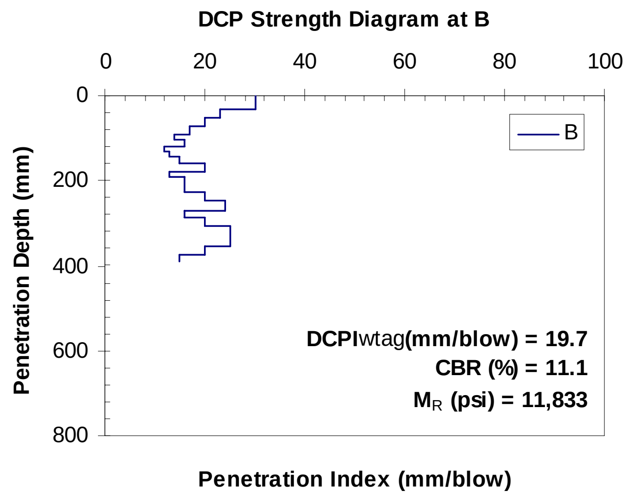


Figure B.1. D16 in Black Hawk County, IA (I.D. No. 12): Rubblization





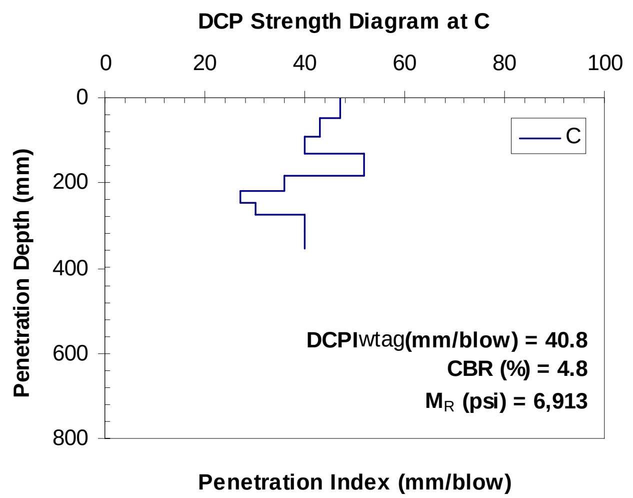


Figure B.2. V43 in Black Hawk County, IA (I.D. No. 13): Rubblization

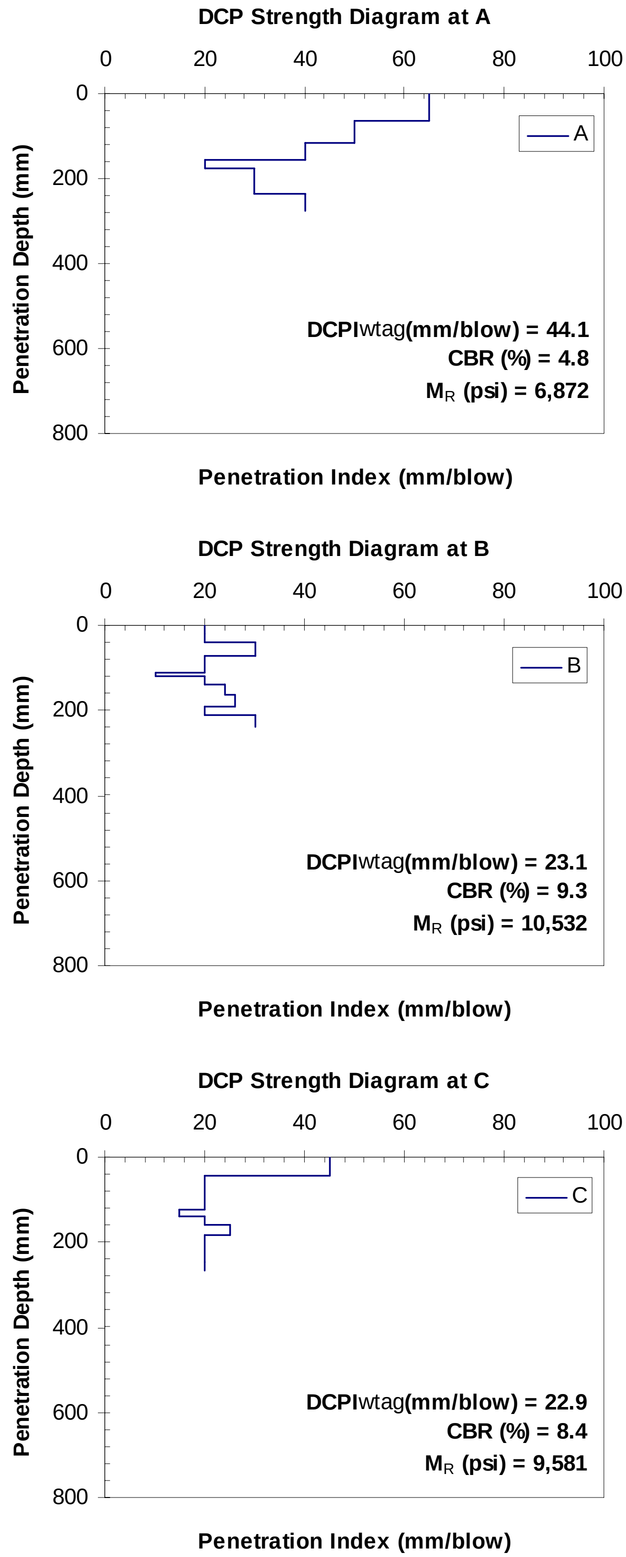
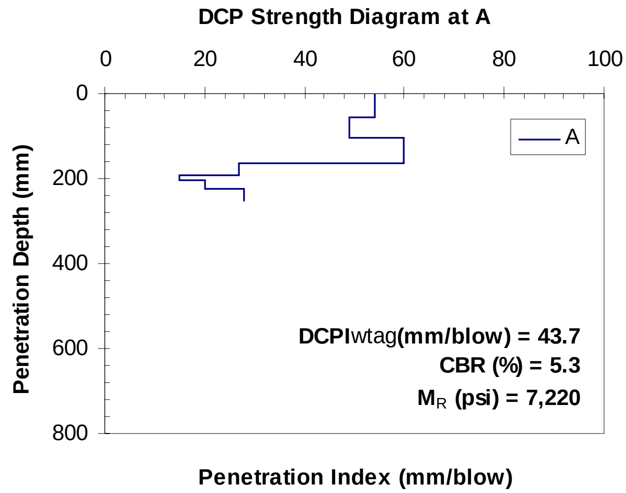
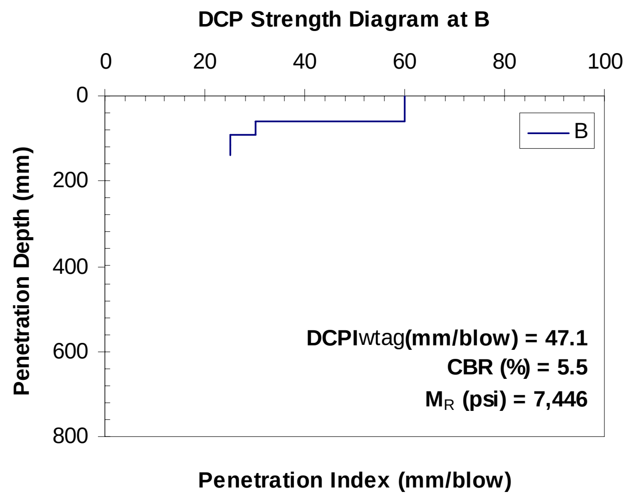


Figure B.3. IA3 in Delaware County, IA (I.D. No. 21): Rubblization





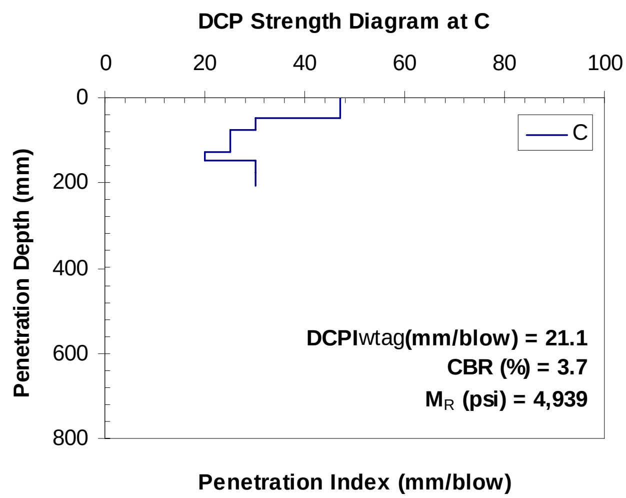
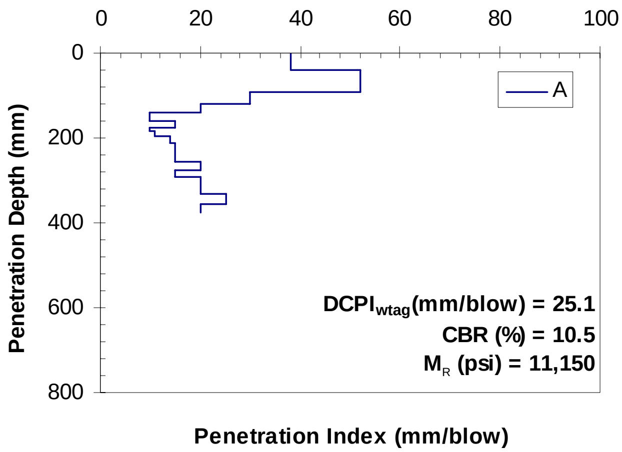
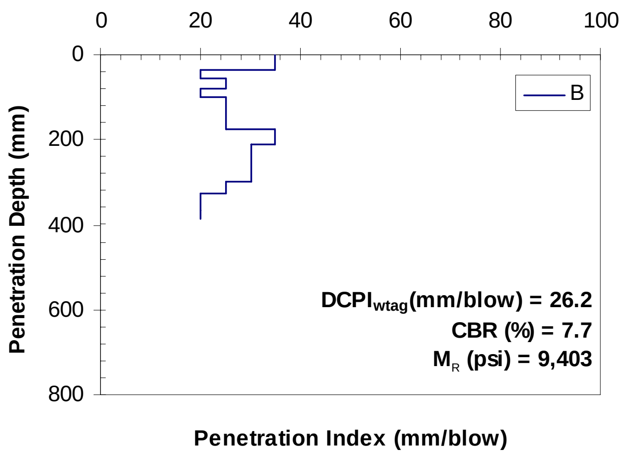


Figure B.4. C23 in Franklin County, IA (I.D. No. 26): Rubblization

DCP Strength Diagram at A



DCP Strength Diagram at B



DCP Strength Diagram at C

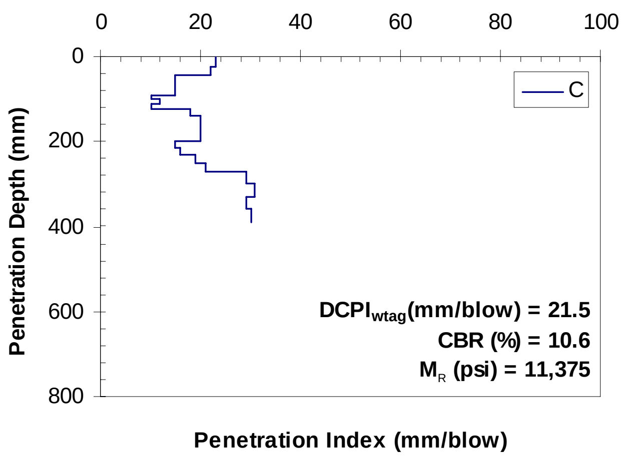
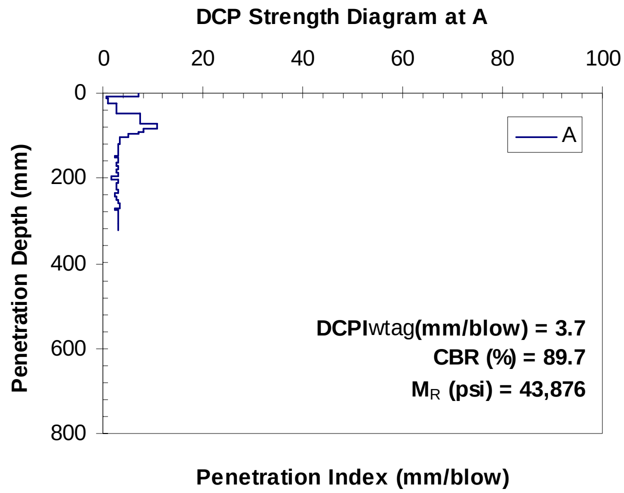


Figure B.5. L55 in Mills County, IA (I.D. No. 32): Rubblization



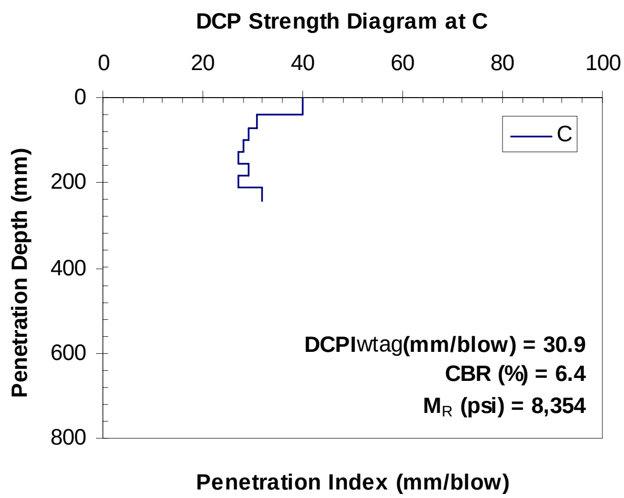
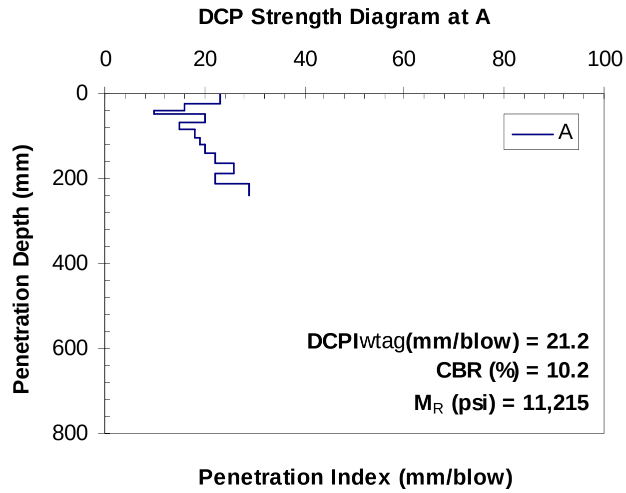


Figure B.6. IA141 in Polk County, IA (I.D. No. 33-1): Rubblization



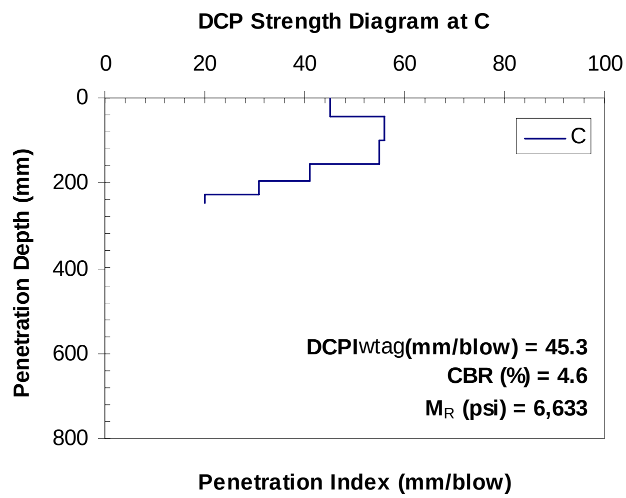
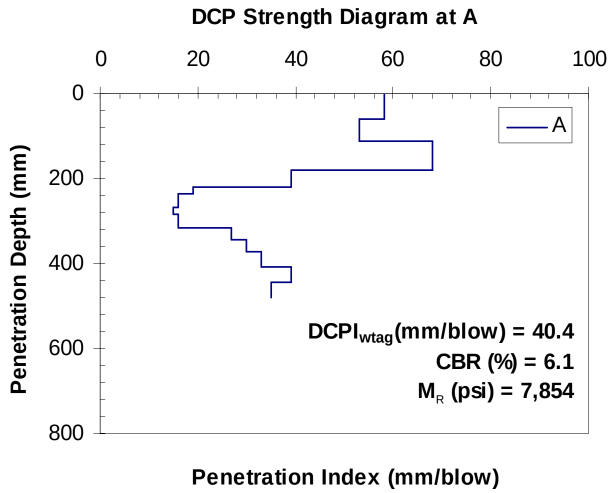
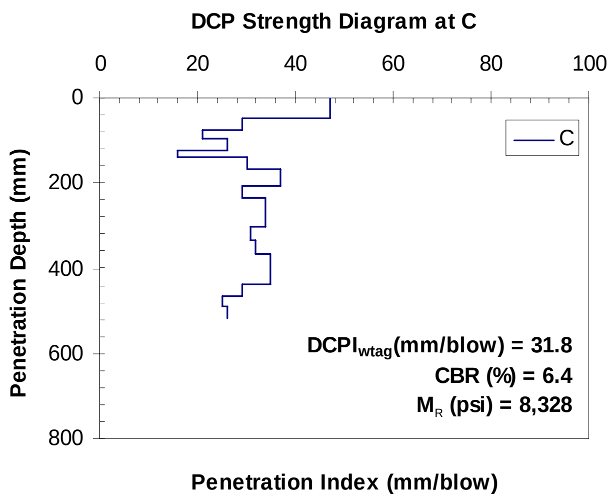


Figure B.7. IA141 in Polk County, IA (I.D. No. 33-2): Rubblization





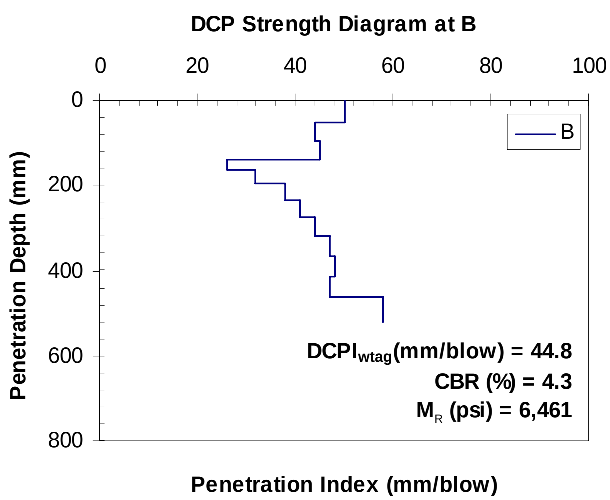
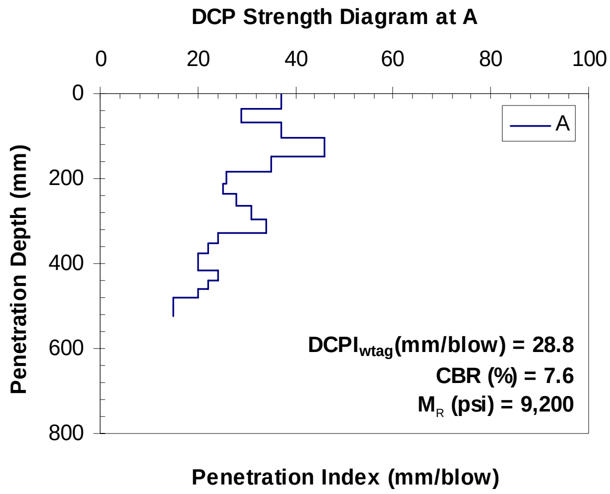
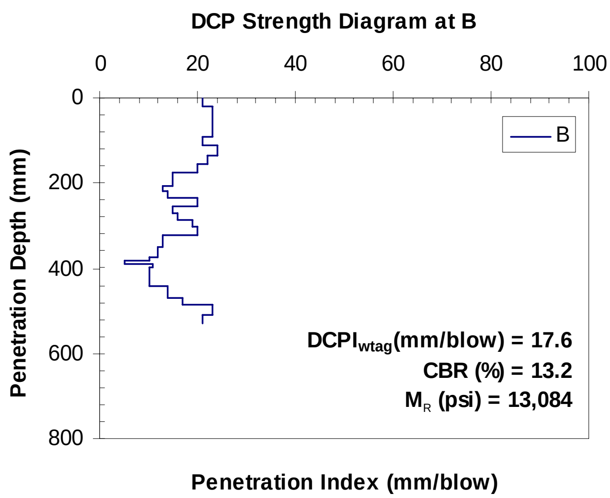


Figure B.8. N72 in Adair County, IA (I.D. No. 26): HMA on PCC





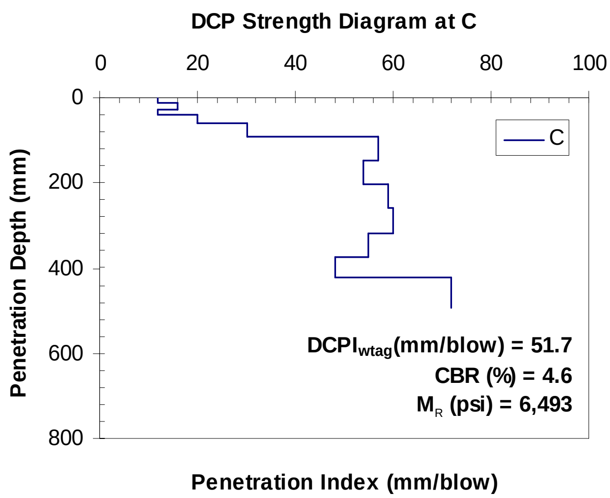


Figure B.9. G61 in Adair County, IA (I.D. No. 2): HMA on PCC

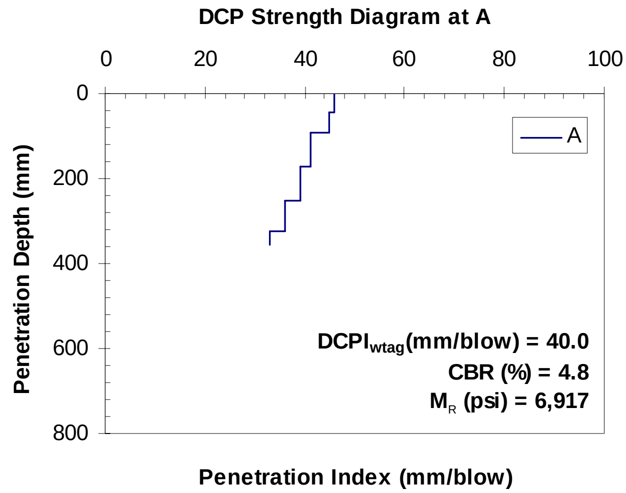
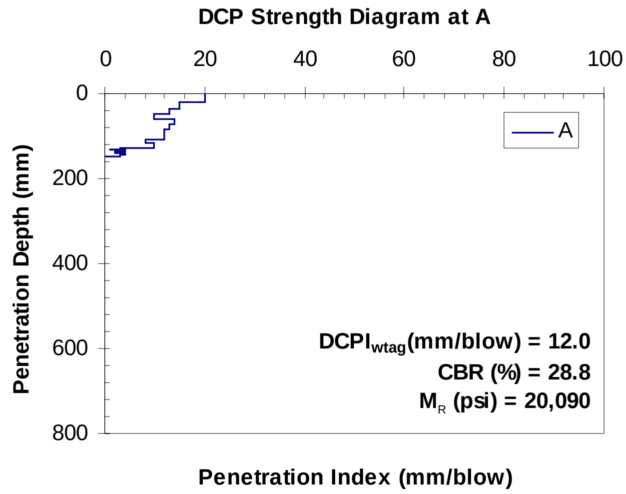


Figure B.10. G61 in Adair County, IA (I.D. No. 3): HMA on PCC



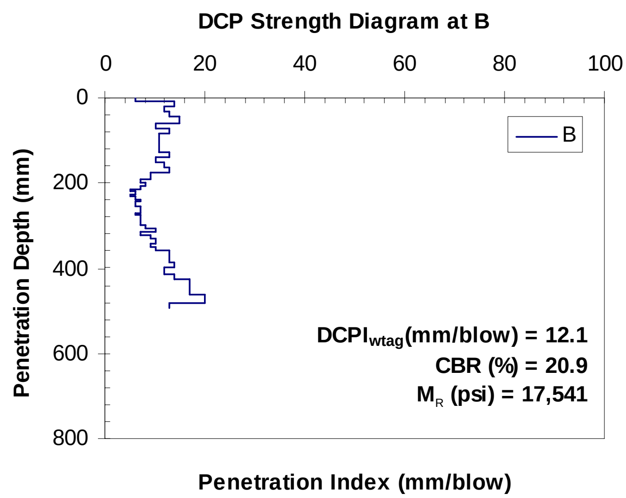
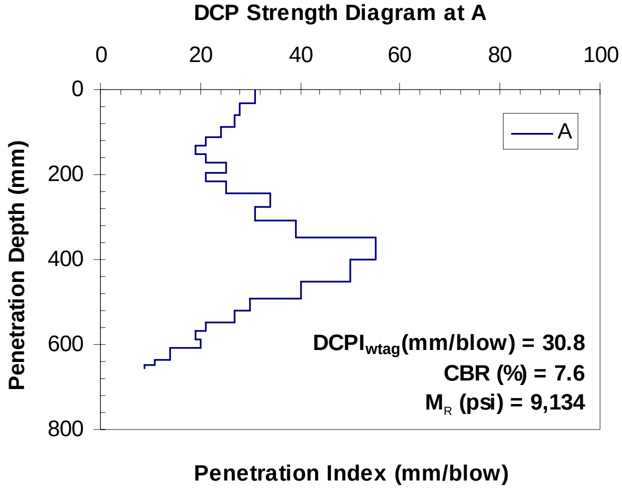
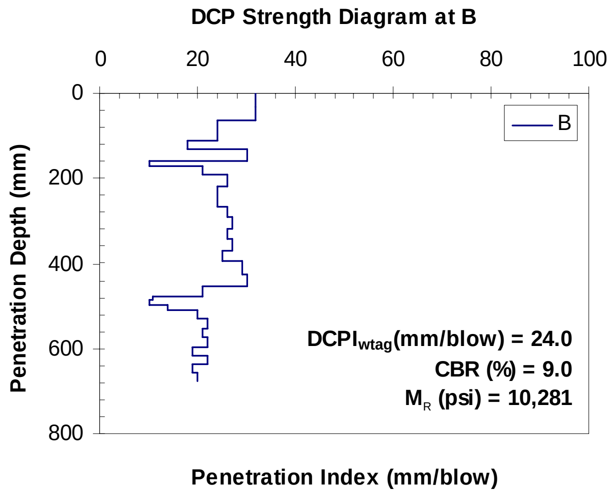


Figure B.11. Tama Road in Black Hawk County, IA (I.D. No. 6): HMA on PCC





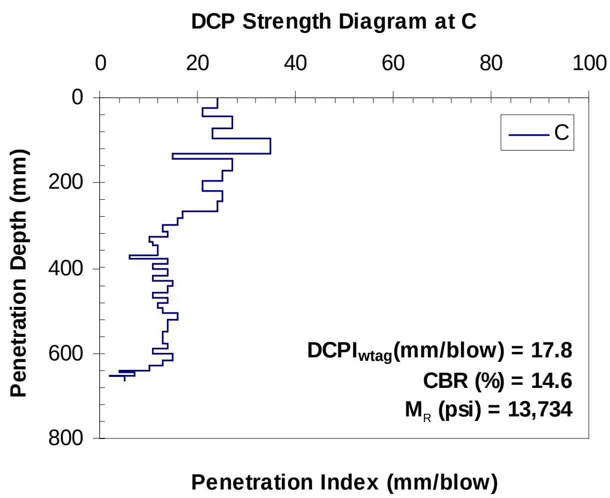
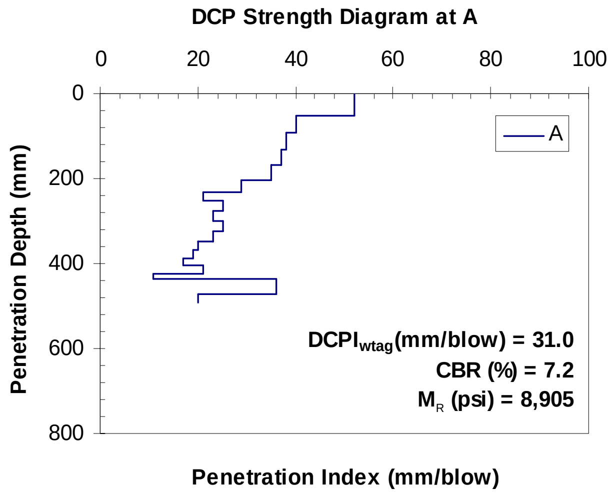
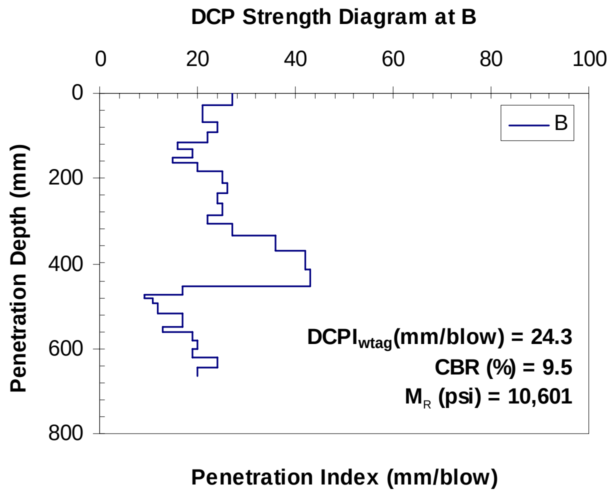


Figure B.12. D52 in Black Hawk County, IA (I.D. No. 8): HMA on PCC





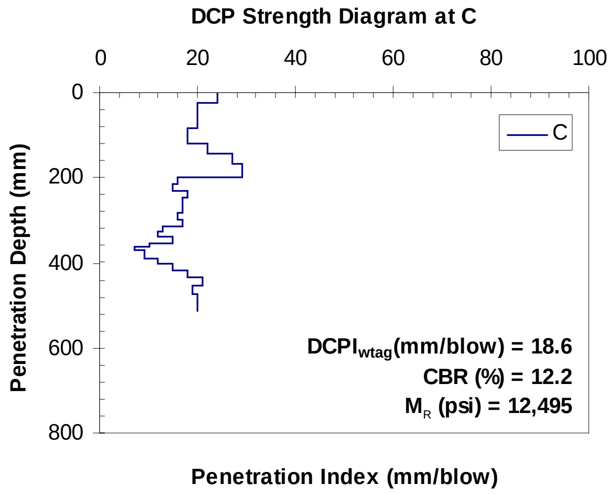


Figure B.13. D38 in Black Hawk County, IA (I.D. No. 14): HMA on PCC

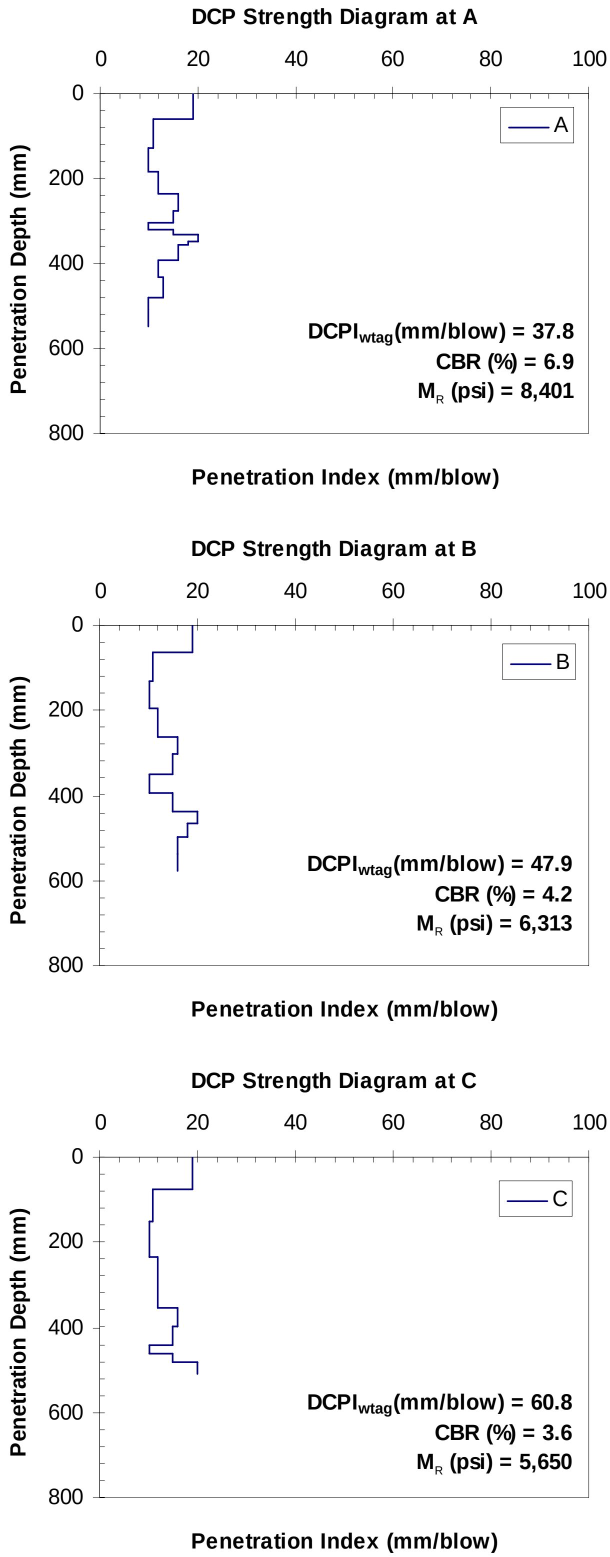
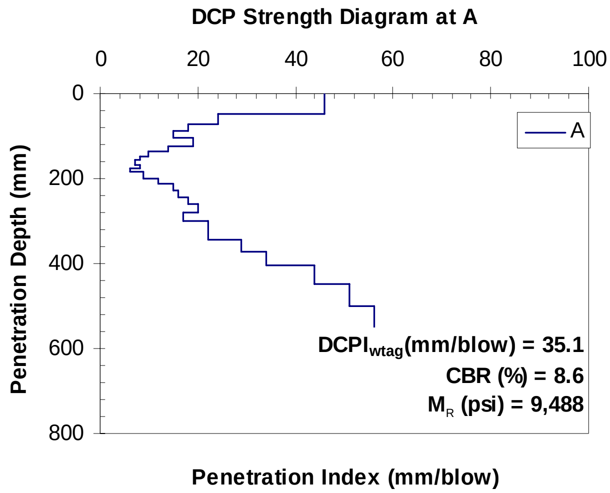
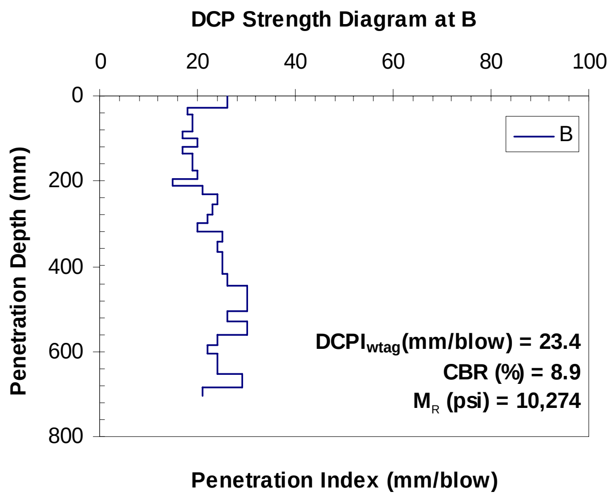


Figure B.14. R35 in Clarke County, IA (I.D. No. 18): HMA on PCC





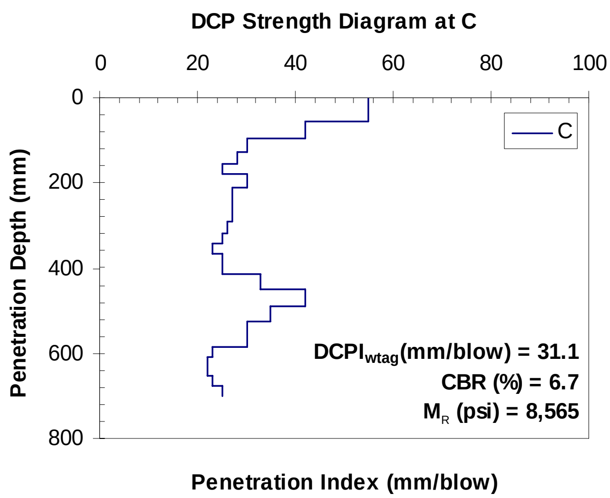
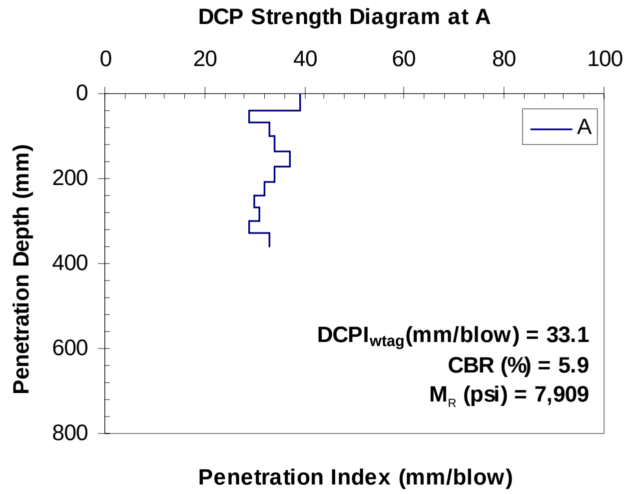


Figure B.15. Hawk Ave. in Grundy County, IA (I.D. No. 28): HMA on PCC



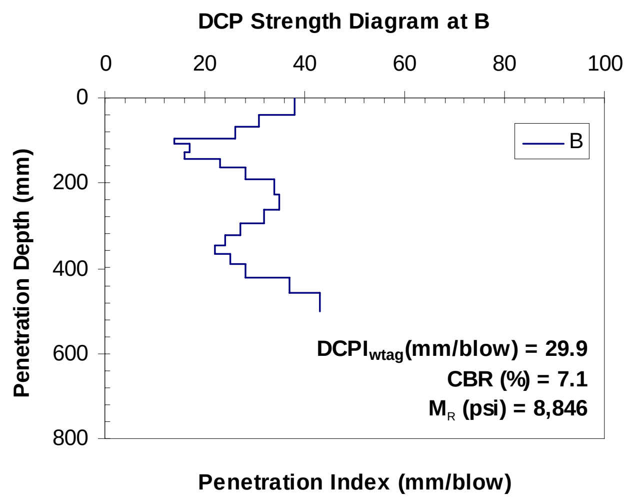
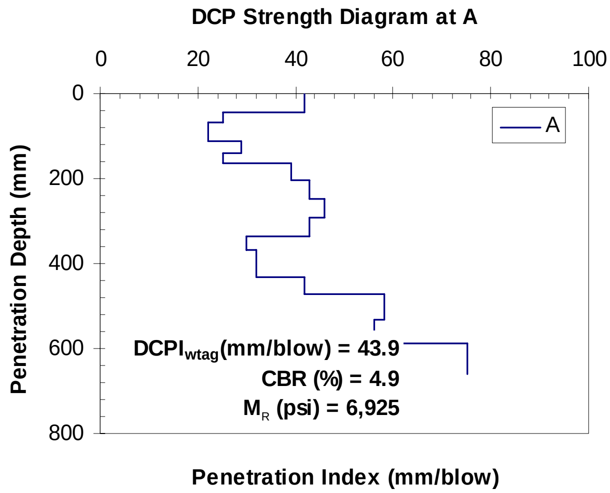
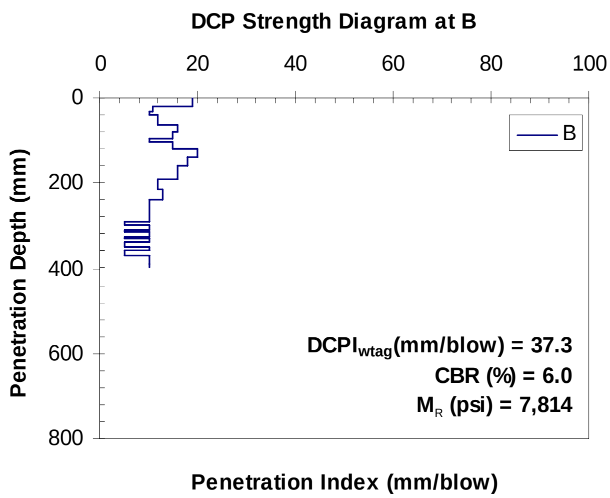


Figure B.16. Old US 6 in Iowa County, IA (I.D. No. 31): HMA on PCC





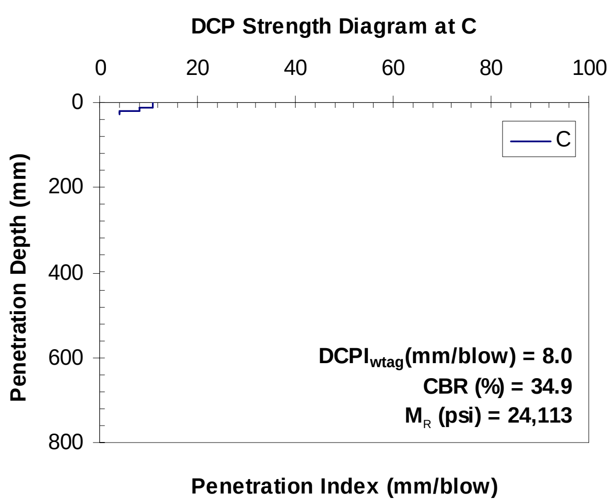
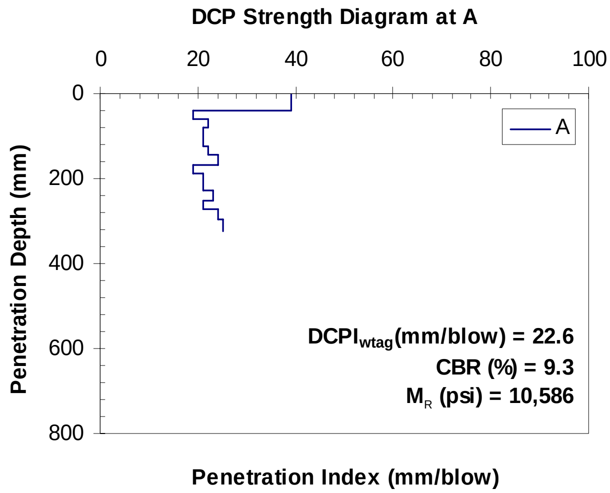
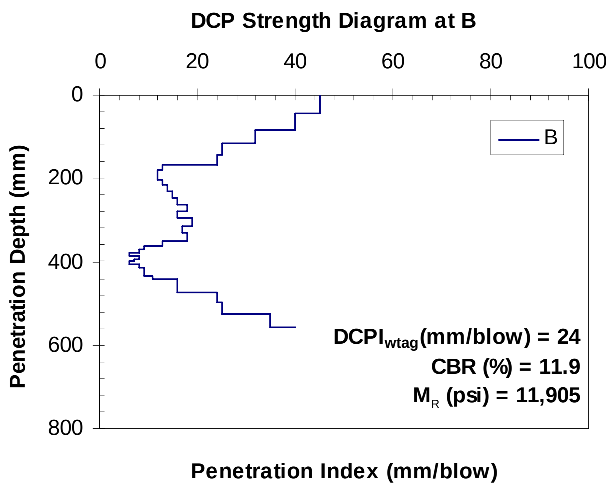


Figure B.17. 4th Ave. in Poweshiek County, IA (I.D. No. 34): HMA on PCC





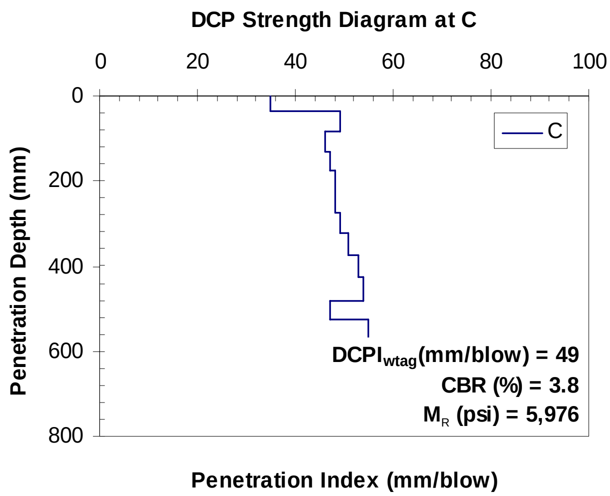


Figure B.18. R50 in Story County, IA (I.D. No. 38): HMA on PCC

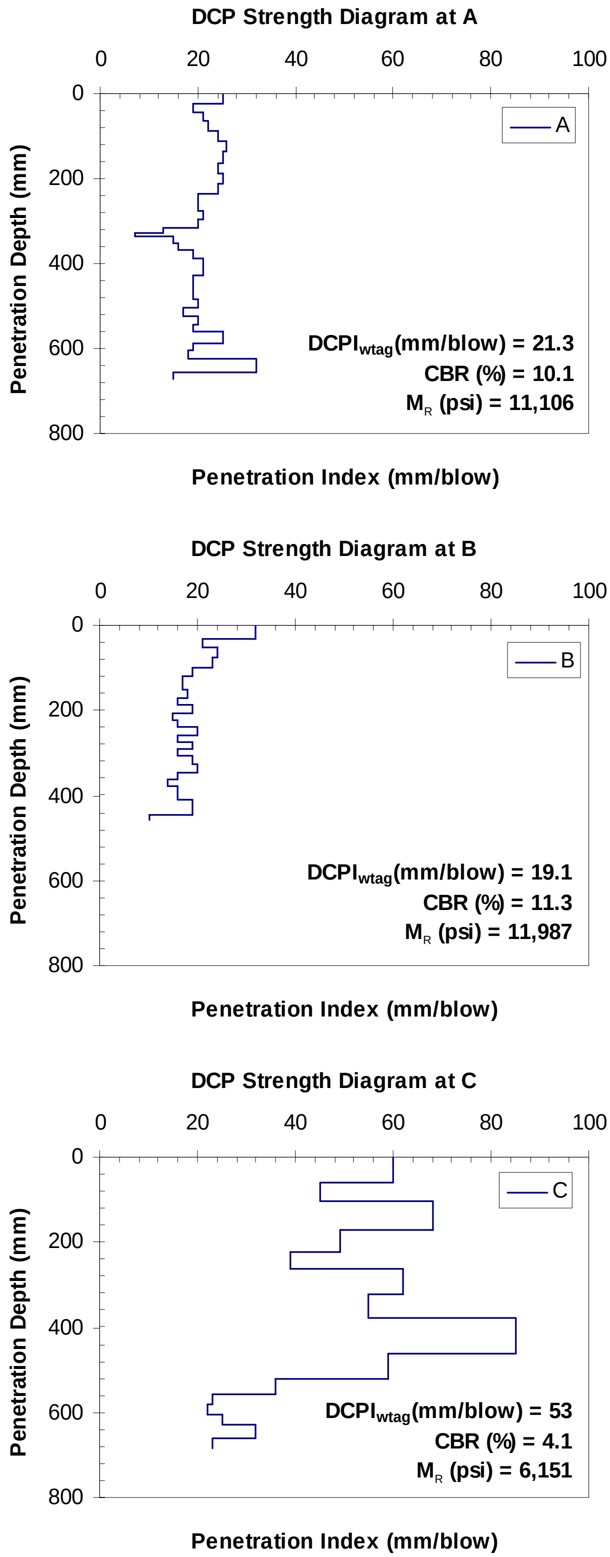


Figure B.19. US63 in Tama County, IA (I.D. No. 39): HMA on PCC

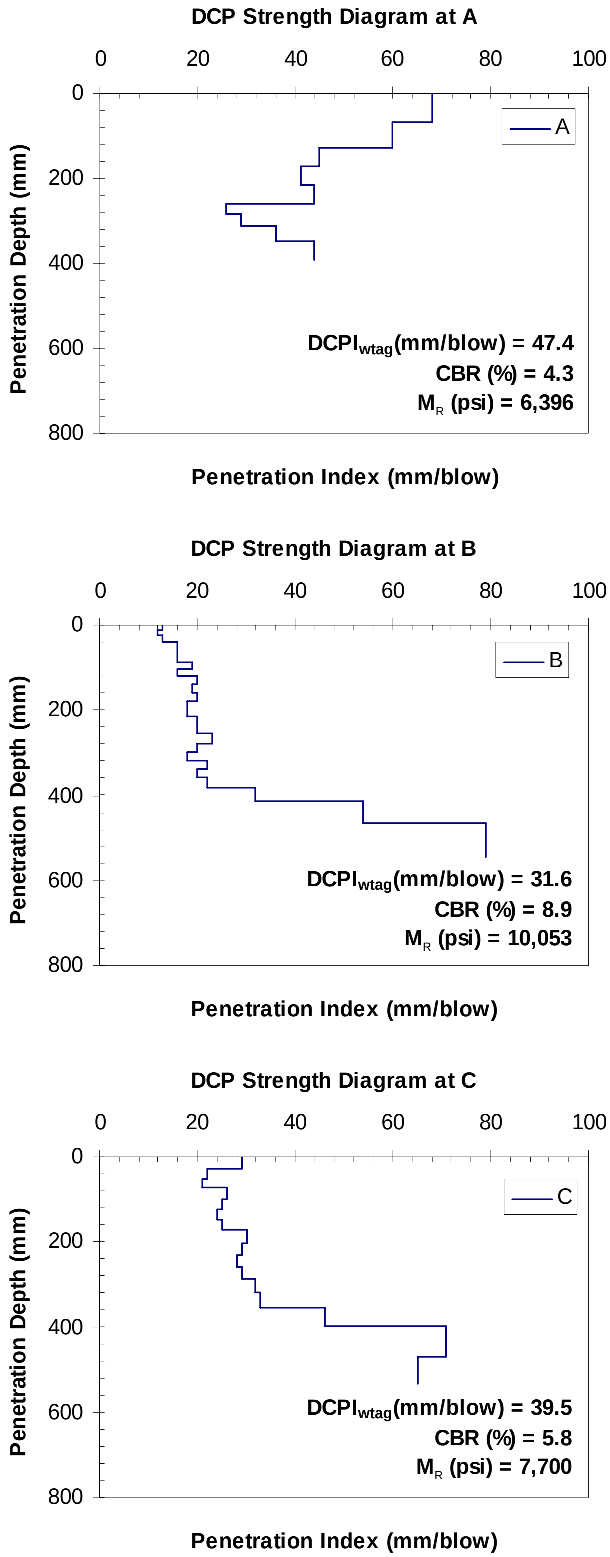


Figure B.20. V18 in Tama County, IA (I.D. No. 40): HMA on PCC

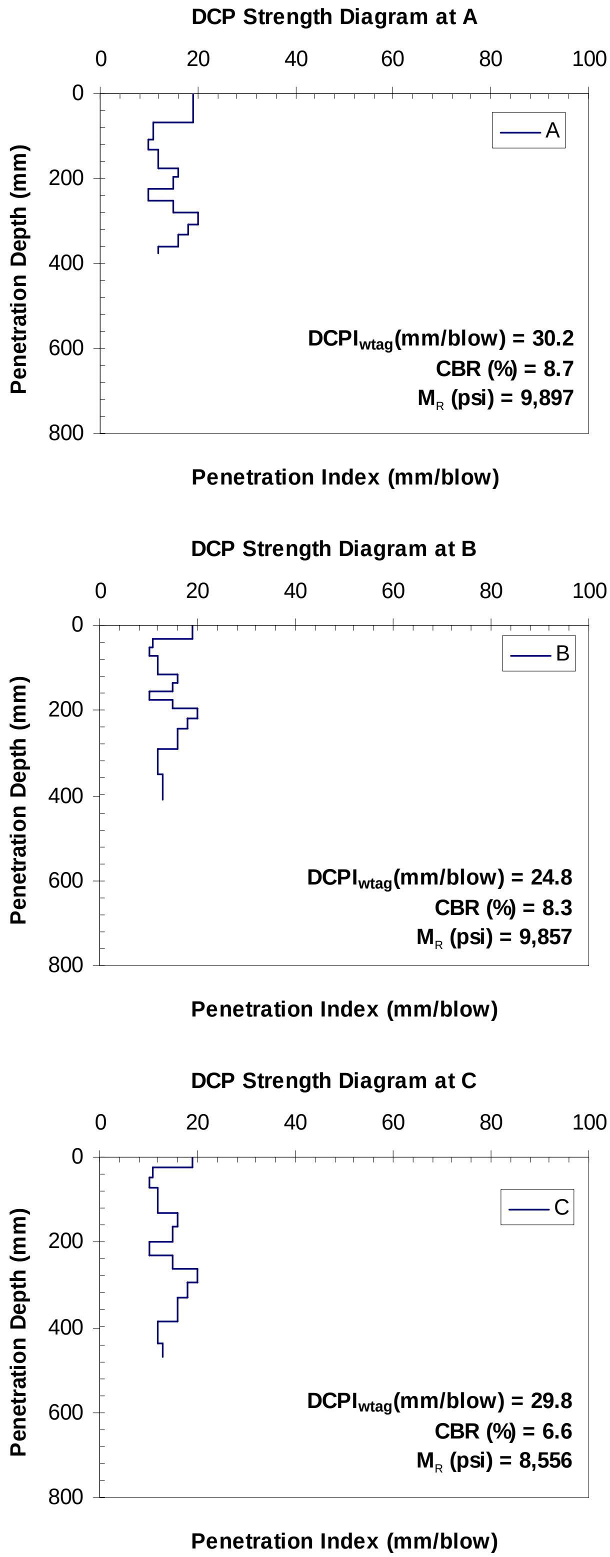
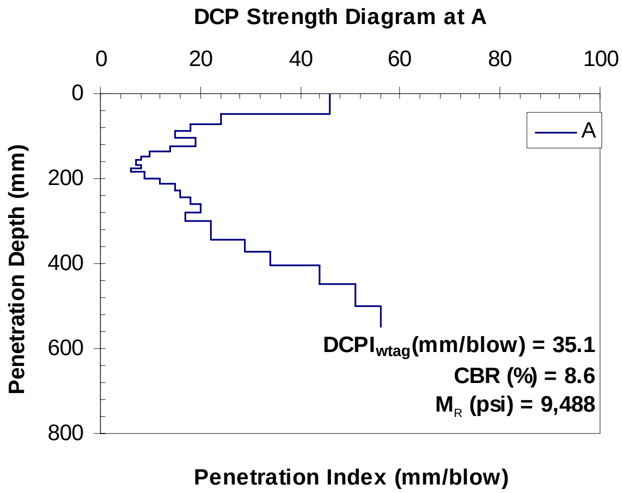
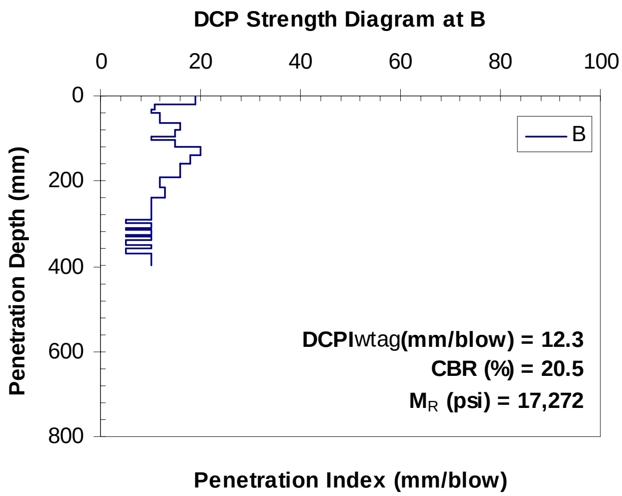


Figure B.21. H24 in Union County, IA (I.D. No. 43): HMA on PCC





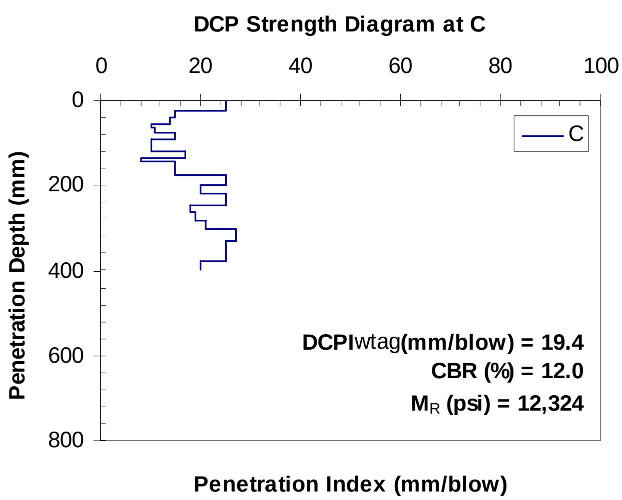
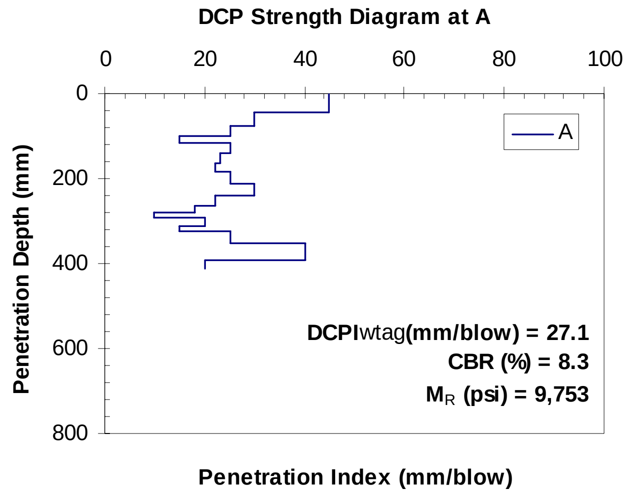
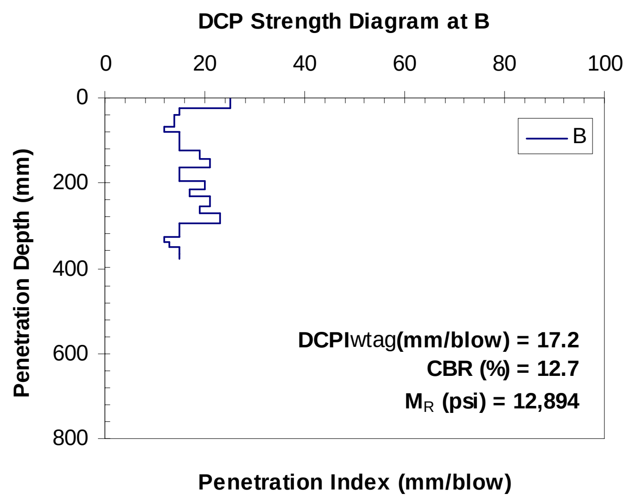


Figure B.22. F33 in Scott County, IA (I.D. No. 35): HMA on PCC





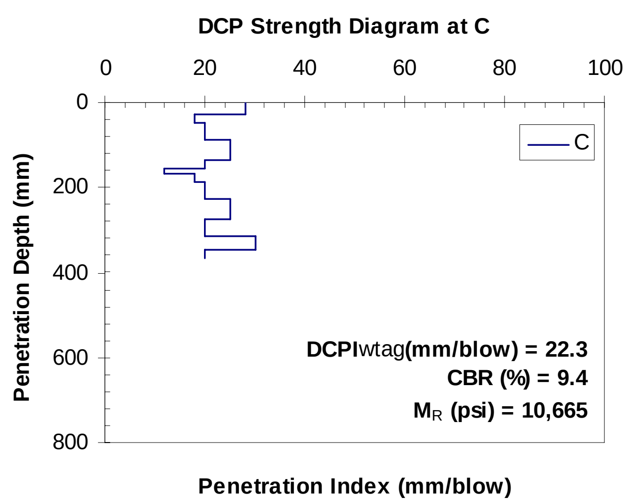
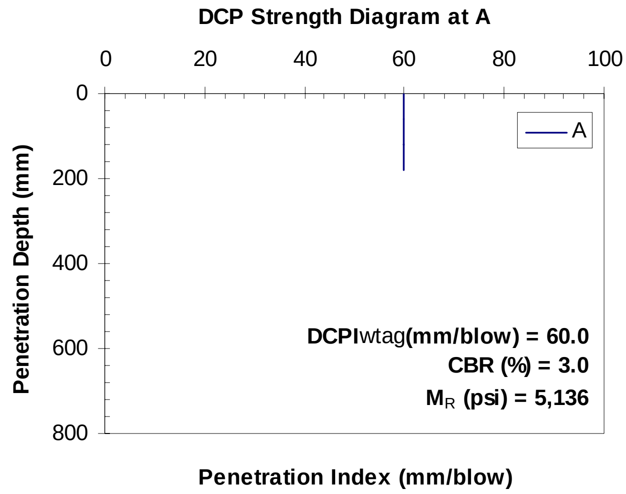
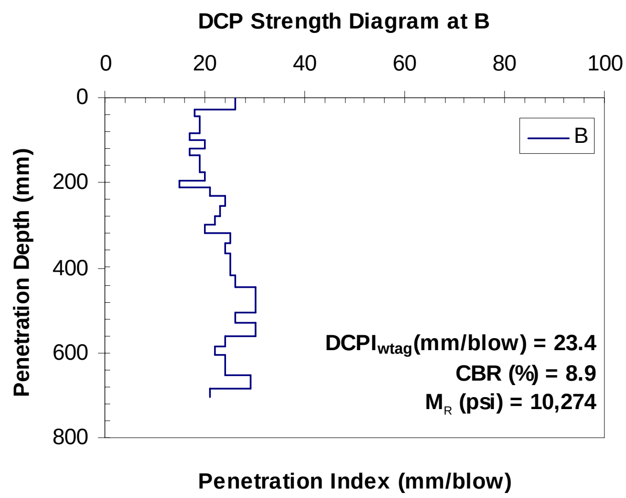


Figure B.23. IA 139 in Winneshiek County, IA (I.D. No. 47): HMA on PCC





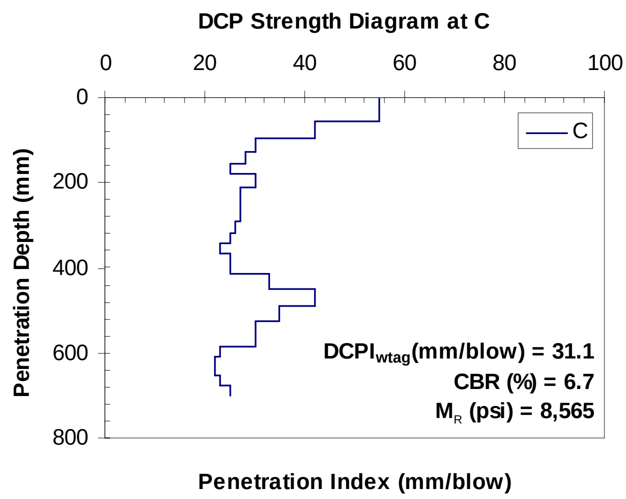


Figure B.24. X16 in Allamakee County, IA (I.D. No. 4): HMA on RAC

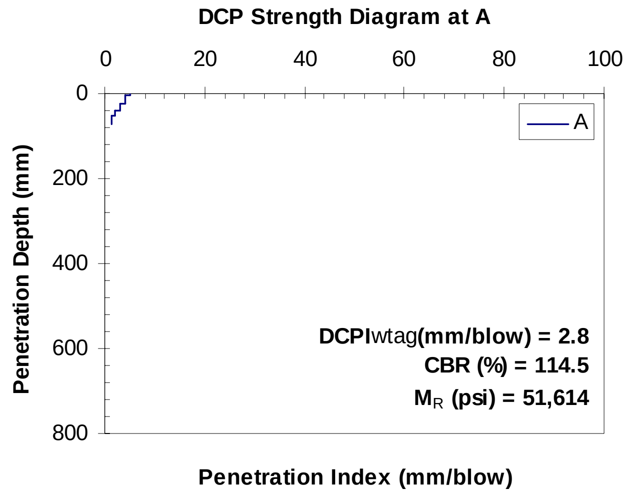


Figure B.25. County Road in Dubuque County, IA (I.D. No. 23): HMA on Aggregate

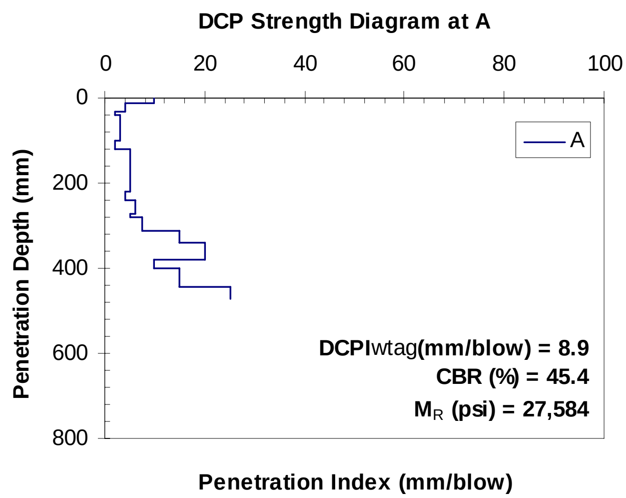
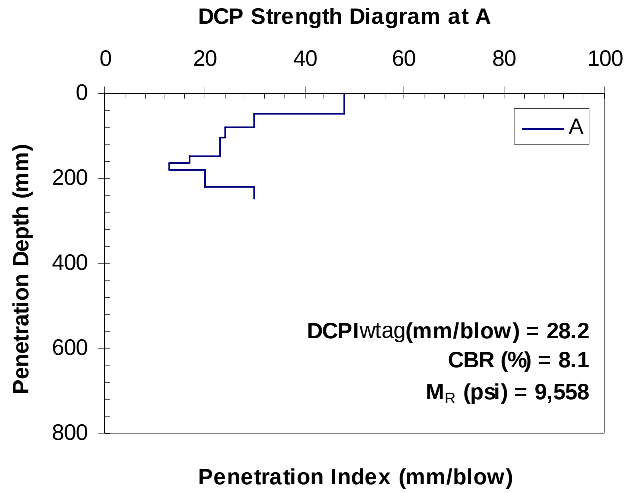
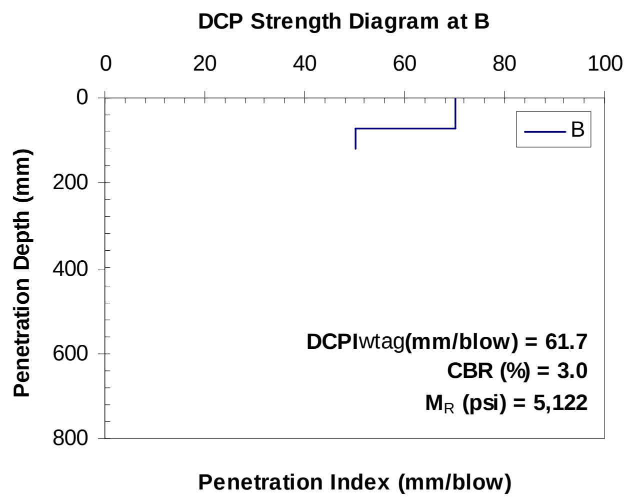


Figure B.26. Local Road in Dubuque County, IA (I.D. No. 24): HMA on Aggregate





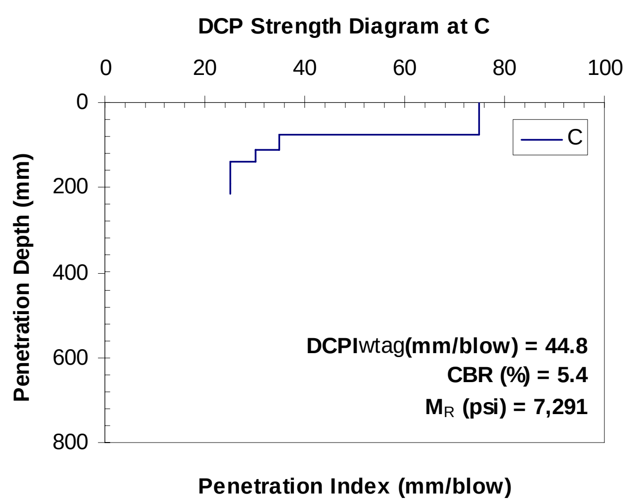
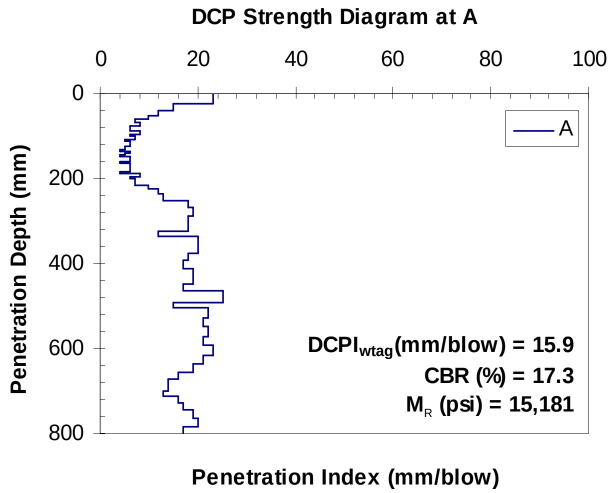
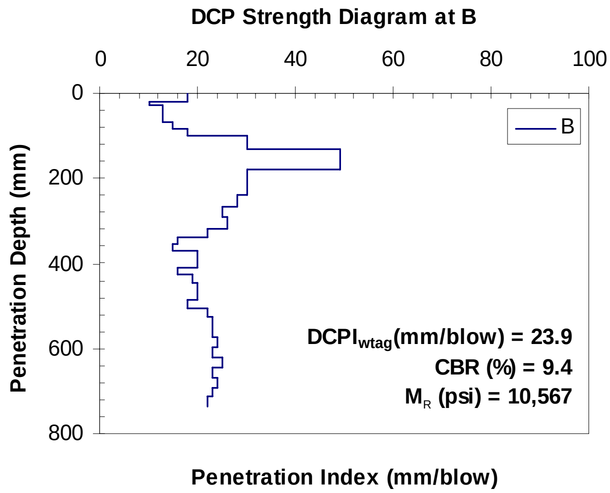


Figure B.27. Twelve Mile Road in Dubuque County, IA (I.D. No. 25): HMA on Aggregate





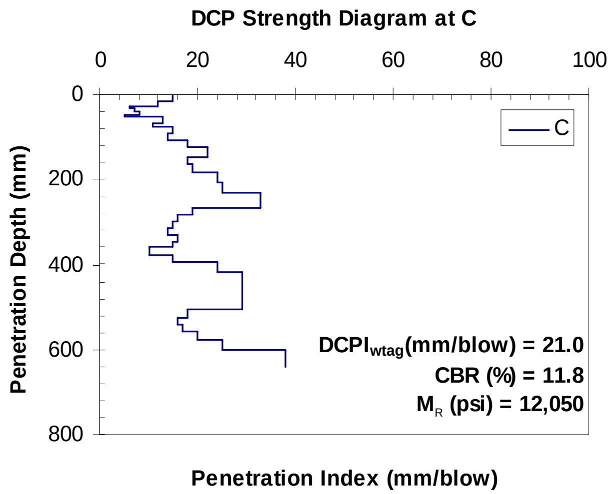


Figure B.28. Multiple Routes in Buchanan County, IA (I.D. No. 17): PCC on HMA

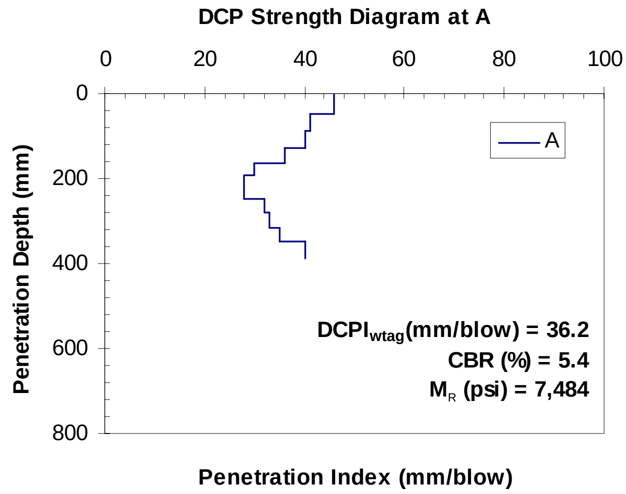


Figure B.29. G24 in Warren County, IA (I.D. No. 46): PCC on HMA

APPENDIX C: VISUAL DISTRESS SURVEY PICTURES

Figure C.1. Overall Pavement Condition on D16 in Black Hawk County, IA (I.D. No. 12): Rubblization

Figure C.2. Pavement Condition on A (Start) of D16 in Black Hawk County, IA (I.D. No. 12): Rubblization

Figure C.3. Pavement Condition at location B (Middle) of D16 in Black Hawk County, IA (I.D. No. 12): Rubblization

Figure C.4. Pavement Condition on C (End) of D16 in Black Hawk County, IA (I.D. No. 12): Rubblization

Figure C.5. Low Temperature Crack on D16 in Black Hawk County, IA (I.D. No. 12): Rubblization

Figure C.6. Overall Pavement Condition on V43 in Black Hawk County, IA (I.D. No. 13): Rubblization

Figure C.7. Pavement Condition on A (Start) of V43 in Black Hawk County, IA (I.D. No. 13): Rubblization

Figure C.8. Pavement Condition on B (Middle) of V43 in Black Hawk County, IA (I.D. No. 13): Rubblization

Figure C.9. Pavement Condition on C (End) of V43 in Black Hawk County, IA (I.D. No. 13): Rubblization

Figure C.10. Block Crack on V43 in Black Hawk County, IA (I.D. No. 13): Rubblization

Figure C.11. Low Temperature Crack on V43 in Black Hawk County, IA (I.D. No. 13): Rubblization

Figure C.12. Overall Pavement Condition on IA3 in Delaware County, IA (I.D. No. 21): Rubblization

Figure C.13. Pavement Condition on A (Start) of IA3 in Delaware County, IA (I.D. No. 21): Rubblization

Figure C.14. Pavement Condition on B (Middle) of IA3 in Delaware County, IA (I.D. No. 21): Rubblization

Figure C.15. Pavement Condition on C (End) of IA3 in Delaware County, IA (I.D. No. 21): Rubblization

Figure C.16. Longitudinal Crack on IA3 in Delaware County, IA (I.D. No. 21): Rubblization

Figure C.17. Low Temperature Crack on IA3 in Delaware County, IA (I.D. No. 21): Rubblization

Figure C.18. Overall Pavement Condition on C23 in Franklin County, IA (I.D. No. 26): Rubblization

Figure C.19. Pavement Condition on A (Start) of C23 in Franklin County, IA (I.D. No. 26): Rubblization

Figure C.20. Pavement Condition on B (Middle) of C23 in Franklin County, IA (I.D. No. 26): Rubblization

Figure C.21. Pavement Condition on C (End) of C23 in Franklin County, IA (I.D. No. 26): Rubblization

Figure C.22. Overall Pavement Condition on L55 in Mills County, IA (I.D. No. 32): Rubblization

Figure C.23. Pavement Condition on A (Start) of L55 in Mills County, IA (I.D. No. 32): Rubblization

Figure C.24. Pavement Condition on B (Middle) of L55 in Mills County, IA (I.D. No. 32): Rubblization

Figure C.25. Pavement Condition on C (End) of L55 in Mills County, IA (I.D. No. 32): Rubblization

Figure C.26. Low Temperature Crack on L55 in Mills County, IA (I.D. No. 32): Rubblization

Figure C.27. Overall Pavement Condition on IA141 in Polk County, IA (I.D. No. 33-1): Rubblization

Figure C.28. Pavement Condition on A (Start) of IA141 in Polk County, IA (I.D. No. 33-1): Rubblization

Figure C.29. Pavement Condition on C (End) of IA141 in Polk County, IA (I.D. No. 33-1): Rubblization

Figure C.30. Longitudinal Crack on IA141 in Polk County, IA (I.D. No. 33-1): Rubblization

Figure C.31. Overall Pavement Condition on IA141 in Polk County, IA (I.D. No. 33-2): Rubblization

Figure C.32. Pavement Condition on A (Start) of IA141 in Polk County, IA (I.D. No. 33-2): Rubblization

Figure C.33. Pavement Condition on C (End) of IA141 in Polk County, IA (I.D. No. 33-2): Rubblization

Figure C.34. Longitudinal Crack on IA141 in Polk County, IA (I.D. No. 33-2): Rubblization