Airfield Asphalt Pavement Technology Program Project 04-01:

Development of Guidelines for Rubblization Final Report

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ABSTRACT

This report presents the results of Airfield Asphalt Pavement Technology Program Project (AAPTP) 04-01, *Development of Guidelines for Rubblization.* The objective was to document the state of the art in rubblization technology for airfield pavements and develop guidelines covering project feasibility, thickness design/material characterization, quality assurance criteria and methods, and construction/equipment issues. Chapter 2 includes an extensive review of the literature on rubblization for both highways and airfields. A comparison of current rubblization specifications is provided, along with a summary of airfield rubblization projects. Chapter 3 presents a suggested criteria and protocol for assessing the suitability of rubblization as a rehabilitation technique if the concrete slabs are less than 9 inches thick and have weak underlying support. Design and construction recommendations are provided when rubblizing these "marginal support" pavements. Chapter 4 covers structural characterization of the rubblized layer, along with other thickness design considerations such as minimum overlay thickness. Chapter 5 deals with quality control and assurance of the rubblization process, providing recommendations regarding test strips, test pits and particle-size acceptance criteria. Future concepts are explored that could utilize in-situ deflection testing or intelligent compaction technology. Chapter 6 provides guidance on equipment and other issues not sufficiently covered in earlier chapters. Finally, Appendix A provides a stand-alone report of the rubblized test sections at the FAA's National Airport Pavement Test Facility (NAPTF), describing the construction, loadings, testing and analysis that were conducted there in 2005. In summary, current technology and practice is adequate for reliably rubblizing airport concrete pavement and overlaying with asphalt.

SUMMARY OF FINDINGS

Chapter 1 - Project Introduction
- Rubblization is the process of fracturing the existing Portland Cement Concrete Pavement (PCCP) in-place into small-interconnected pieces that serve as a base course for a new Hot Mix Asphalt (HMA) overlay. Since there are no hauling or disposal costs and all of the existing pavement materials remain in-place, rubblization is a very cost-effective rehabilitation method. The net effect of rubblizing is converting a deteriorating rigid pavement system into a new flexible pavement system with no reflective cracking concerns.

Chapter 2 – State of the Practice Review
- Excellent industry manuals on rubblization include AI's MS-17 and NAPA's IS-132.
- There are no major flaws in the FAA's EB-66 and spec (P-215) on rubblization.
- TRB Circular No E-C087 (2006) is an excellent consortium of six papers on rubblization.
- Performance studies by LA, CO, AL, IL, MI and other highway agencies all show rubblization to be an effective and efficient rehabilitation alternative.
- Of the 50 million SYs of PCCP rubblized, roughly 2% has been airfields.
- With an exhaustive literature review, there was no documented mention of reflective cracking ever occurring from underlying rubblized PCCP on any highway or airfield project.
- There have been about 30 airfield rubblization projects to date, listed in Tables 2.1 and 2.2 with related project information. The PCCP slab thicknesses of these 30 projects have ranged from 6 to 26 inches.
- The Multi-Head Breaker (MHB) and Resonant Pavement Breaker (RPB) have fairly equally shared the rubblization market for both highways and airfields, demonstrating that both types of rubblization equipment can be effective under a wide array on in-situ conditions.
- A comparison of current rubblization specifications is provided in Section 2.5.

Chapter 3 – Assessing Pavement Suitability
- The combination of thin slabs (less than 9-inches) <u>and</u> weak underlying support (little to no subbase and soft subgrade that is often saturated) can produce marginal stability and difficulty in effectively rubblizing the PCC to specified criteria. World War II era airfields built in the U.S. and later converted into GA airfields often fit this category. Of the 30 airfield rubblization projects, 13 had slabs between 6-8 inches thick. On three of these, significant stability problems were reported while the other 10 apparently went well.
- Typical problems that can occur on these "marginally stable" pavements are large unbroken PCC pieces (with both MHB and RPB), rutting and "punch-thrus" by the RPB, shifting and rotation of large PCC pieces by the HMA haul trucks and pavers (with both MHB and RPB). All these lead to full-depth patching of the rubblized and underlying layers.
- This research recommends adopting the TxDOT evaluation framework (with slight adjustments) in conjunction with a risk assessment chart modified from ILDOT criteria. The background, rationale, protocol and details of this recommended assessment protocol for airfields is explained in Section 3.2. Pavements with a substantial subbase (or slabs ≥ 9-inches) will not likely be problematic.
- Even when a significant portion of a project is assessed with a moderate to high level of risk, rubblization may still be the most attractive and economical rehabilitation alternative due to

its many advantages. If so, there are a number of recommendations and considerations in the planning, design and construction stages to optimize success. These include:
- installing edge drains
- conducting a feasibility trial
- allowing "modified rubblization" (temporarily waiving particle size criteria)
- considering conventional crack and seat
- including an aggregate leveling course
- avoiding wet seasons
- including a separate bid item (and estimated quantity) for full-depth patching
- turning roller vibrators off during the rubblization process
- using tracked pavers for the first HMA lift
- sequencing HMA haul operations to avoid trafficking on the rubblized material

Each of these recommendations for rubblizing on marginal pavement structures is covered in detail in Section 3.3.

Chapter 4 – Material Characterization and Other Thickness Design Considerations
-   To determine appropriate design modulus values for a rubblized layer, the literature was reviewed for studies where back-calculated modulus values of rubblized layers (E_{rub}) were determined. In addition, new back-calculations were performed for several projects, providing a total of 17 unique rubblized sections. The average in-place E_{rub} value was 205 ksi, with the range being 100-430 ksi. This is significantly stiffer than that of a crushed aggregate base, which is assumed to have a modulus of 50-60 ksi.
-   Rubblized modulus appears to be influenced by slab thickness (t), although there was a low R^2 correlation factor (0.32) between E_{rub} and t. Thicker slabs tended to have higher E_{rub}.
-   The concept of "Retained Modulus" was explored. Retained Modulus is defined as the ratio of E_{rub} to pre-fractured PCC modulus, or $E_{rub} / E_{pcc.}$ There were 13 sections (of the 17 sections) with sufficient data for determining $E_{rub} / E_{pcc.}$ Retained Modulus values were in the range of 1.8-13.5%, with an average of 6.0%. The thicker slabs tended to have higher E_{rub} / E_{pcc} values versus thinner slabs. The R^2 correlation factor between (E_{rub} / E_{pcc}) and (t) was fair (0.69). The concept of Retained Modulus needs to be explored further, but seems to make sense considering that the presence of reinforcing steel, hard aggregates, etc should have the same positive influence on both pre and post-rubblized modulus.
-   There was an indication on four separate projects that E_{rub} appeared to increase over time, albeit not at a consistent rate. While interesting, we do not recommend this trend be considered when determining E_{rub} design values.
-   Rubblized modulus seems to be dependent on the level of rubblization. Repeat runs of either rubblization equipment were found to reduce E_{rub} as the concrete became more fractured. Thus, the fracture criteria specified, and its enforcement, can have an effect on E_{rub}.
-   Combining all the findings summarized above leads to no precise method for predicting E_{rub} during the design process. That being said, it does appear E_{rub} is somewhat related to slab thickness. This research team recommends the following design ranges on airfield projects:
    - For slabs 6 to 8 inches thick: E_{rub} from 100 to 135 ksi
    - For slabs 8 to 14 inches thick: E_{rub} from 135 to 235 ksi
    - For slabs >14 inches thick: E_{rub} from 235 to 400 ksi

- When the CBR method of pavement design is used, characterizing the rubblized layer as a crushed stone base (CBR=100) is conservative. We recommend using the following CBR equivalency factors (EF) to convert from rubblized to an "equivalent" crushed stone base:
  - Rubblized Layer 6 to 8 inches thick: EF = 1.2
  - Rubblized Layer 8 to 14 inches thick: EF = 1.4
  - Rubblized Layer > 14 inches thick: EF = 1.6
- Heavily reinforced slabs can cause concern regarding effectiveness of rubblization, as there can be significantly less fracture below the steel. That being said, there was no mention in the literature of any rubblized projects where reflective cracking occurred in the HMA due to poorly fractured PCC. Full depth fracture must be confirmed in the test pits.
- There was no noted change in subgrade moduli values before and after rubblization.
- There were no obvious trends or consistent differences between E_{rub} produced from the MHB and the RPB. Both types of equipment at different times and on various sections produced higher, lower, and similar E_{rub} relative to the other.
- The <u>minimum</u> HMA overlay thickness is 5 inches if placing directly on rubblized. This must be placed in a minimum of two lifts, with the first lift at least 3 inches thick in order to achieve density. Unbound material such as P-209, RAP, etc is often placed first on rubblized as a leveling course for runways and taxiways to reestablish grade. If an unbound layer is placed directly over rubblized, the minimum unbound layer thickness is 4 inches, and the minimum HMA overlay thickness criteria for that particular material should apply.

Chapter 5 – Quality Assurance Practices and Future Concepts
- The purpose of a test strip and test pit is for contractor to optimize his equipment operation and then demonstrate effective rubblizing and rolling practices for the in-situ conditions.
- The test strip location needs to be representative of the entire pavement feature to be rubblized. A new test strip (and test pit) should be performed when different conditions of slab age and thickness, reinforcing steel, support layers and thicknesses, and subgrade or moisture conditions are encountered.
- The dimensions of the test strip should be 300 ft long by one slab width.
- Any rolling (including proof rolling) performed on the project should be part of the test strip.
- A test pit is excavated within each test strip to verify fracture was full depth and to the conditions specified, namely with regard to particle size and steel debonding.
- Desired test pit dimensions are the full slab width of the test strip by at least 6 ft long.
- Locate the test pit so that it includes a transverse joint and a longitudinal joint.
- Avoid locations that are near the wheel-path of normal aircraft traffic.
- Examine test pit rubble to determine if specification criteria have been met, including:
  - fracture through full depth of PCC slab
  - steel "substantially" debonded from concrete (dowels may be sawed at the joint if can't achieve "substantial" debonding)
  - particle size requirements are met
  - no contamination of subgrade fines into rubblized layer
  - no shear distortion of the subgrade
- If the test pit is located along a free edge, the sizes of the broken PCC pieces may be larger and "out of spec" at the outside edge (2-3 times slab thickness) due to the lack of lateral support during rubblization. This "out of spec" should be allowed.

-   Repair test pits by carefully discarding all excavated material and using high quality crushed aggregate or HMA as patch material. Compact in necessary lifts.
-   The recommended particle size acceptance criteria is:
    - Upper half of slab: No particles > 6 inches in any dimension, and at least 75% of material (by weight) < 3 inches in any dimension
    - Bottom half of slab or below steel: Virtually no particles > than 2X the slab thickness, up to 24 inches, in any dimension.
-   On the rare occasion of an oversized piece, leave the piece in-place versus trying to remove.
-   Future concepts for improving current quality control/assurance practices include utilizing either deflection testing or intelligent compaction to identify potential unrubblized areas, find localized weak spots, and monitor the uniformity of the rubblized and other support layers.

Chapter 6 – Other Items

-   In general, we recommend the installation of a longitudinal edge drain system for airfield rubblization projects and that it is operating at least two weeks prior to the start of rubblization. It is important that the edge drain be placed next to where the rubblized layer will be constructed and extends to the top of the rubblized layer.
-   The project plans should identify items that need to be protected from rubblization such as existing utilities, electrical fixtures, drainage inlets, adjacent pavements and other structures. The contractor is then responsible to operate his equipment in a manner that will not damage those items. Full depth isolation saw cuts are typically used as a means of protection.
-   The equipment descriptions provided in the FAA's P-215 and the Air Force's ETL 01-09 for the MHB is current. The description of the RPB in these two specs needs updating to allow for variation in frequency and greater amplitude. Any description should include "self-contained, self-propelled" to rule out the use of impact roller devices (i.e. *Impactor 2000),* wrecking balls, etc. These devices were developed for demolition and removal, and should never be used for rubblization. Both the MHB and the RPB have demonstrated they can be successful under a wide range of in-situ conditions.
-   A spec should allow the use of relief trenches as the contractor deems appropriate.
-   All HMA must be removed prior to rubblizing the PCCP with either type equipment.
-   Finish grading of the rubblized surface, for smoothness or grade control, is not possible. Milling of the PCCP prior to rubblization for grade control has been done.
-   Applying a prime or tack coat on the rubblized surface is not recommended.
-   A windrow elevator to feed the hopper of the paving machine on the initial lift of HMA should not be allowed because of the risk of disturbing/dislodging particles at the surface.
-   A separate bid item with estimated quantity should be included for full-depth patching.
-   It is best to keep traffic away from the rubblized pavement until all work is completed.
-   The cost of rubblizing has been between $1.50 to $2.50 per SY when both types of rubblization equipment is allowed. If only one type is allowed, the price range has been slightly higher at $2.00 to $3.00 per SY.
-   With regard to the concern that a PCCP suffering from Alkali-Silica Reaction (ASR) could potentially expand after rubblization to cause future differential swelling and subsequent roughness on the pavement surface, we found no mention of this concern in the literature or in speaking with rubblization experts. In addition, there has been a long history of highway and airfield projects utilizing fractured slab technology on ASR-infected PCCP and their long-term performance has been excellent.

CHAPTER 1 – PROJECT INTRODUCTION

This chapter presents the background to this project, explains the approach taken by the research team and provides an overview of this report.

1.1　Background

Rubblization technology and equipment were initially developed to rehabilitate aging highway pavements; however, there is growing interest in using this technology for rehabilitating aging airfield pavements as well. While the use of rubblization on airfield Portland Cement Concrete Pavement (PCCP) has increased over the past few years, it remains a relatively new technology for airfields compared to highways. To quantify, of the approximate 50 million square yards of PCCP that has been rubblized on hundreds of projects in the U.S. this past decade, 98% has been on highways and only 2% on airfields [Buncher, 2006]. At the time of this writing, there have only been approximately 30 airfield projects utilizing rubblization, with the majority occurring in the past five years. These projects covered a wide range of airfield types, from small general aviation to heavy load military; and a wide range of geographic locations, from Florida to Washington State.

The widespread use and success of rubblization on highways provides the basis for tremendous payback potential when applying this technology to airfields. Within the FAA Integrated Airport System airfield infrastructure and the U.S. Defense Department airfield inventory, there are more than 83 million square meters of PCCP greater than 13 inches thick and more than 35 years old. There are also many WWII era airfields built by the US Army with PCC thicknesses that are less than 8 inches and have no underlying subbase. Many of these have been converted into local General Aviation Airfields (GAA) and these aging PCC pavements will need major rehabilitation within the next 10 years.

While rubblization has been deemed successful on the overwhelming majority of airfields projects, there are still many questions that remain when applying this technology to airfields. Airfield pavement conditions can be significantly different from those of typical highway pavements, where rubblization has the longer history of use. These differences need to be addressed with regards to how they affect the rubblization process. One example pointed out in the last paragraph is slab thickness. Highway PCCP thickness typically ranges between 8 to 12 inches, while airfields can be as thin as 6 inches or as thick as 25 inches. Separate challenges exist for rubblizing the very thin slabs and the very thick slabs that do not exist for typical highway thicknesses. Another area where specific guidance for airfield rubblization is critical is in thickness design, since procedures for designing airfields varies greatly from traditional empirical highway procedures.

State agencies that use rubblization each have their own set of policies, procedures and specifications developed from that state's experiences with their set of conditions. This guidance often varies considerably from state to state, which is understandable considering pavement design methods, materials and construction practices vary from state to state. Even the guidance of individual states will change frequently as more and more experience is gained with rubblization technology. For this project, the challenge is to develop rubblization guidelines that are appropriate for the vast multitude of conditions that may exist on airfields across the country.

The objective of this project is to document the state-of-the-art in rubblization technology for airfields and to develop guidelines covering project feasibility criteria and protocol, thickness design considerations such as material characterization, quality assurance criteria and methods, and construction/equipment issues. The successful completion of this project will further develop rubblization technology for civilian and military airfield pavements resulting in improved rehabilitation decisions and overall project quality.

1.2   Approach and Report Overview

Chapter 2 provides substantial detail covering the extensive literature review that was initially conducted under this project to collect and synthesize information relative to rubblization of PCCP with special emphasis on airfields. This involved a review of the existing state of practice to include the technical literature such as industry manuals, published papers, reports and other documents. Interviews were also conducted with rubblization contractors, hot mix asphalt contractors, design consultants and highway agencies which have all been heavily involved in rubblization projects. In addition, Chapter 2 includes a review and discussion of individual airfield rubblization projects. These projects cover a wide range of site and load conditions, as well as design approaches. Successes, failures and lessons learned are especially noted. Finally, a review of rubblization specifications was conducted from the many highway agencies that routinely use rubblization, and a comparison of key features is made between these state specs and the current Air Force and FAA rubblization specifications being used.

Two areas of great uncertainty for engineers considering the use of rubblization on an airfield rehabilitation project are in regards to what conditions should rubblization be utilized and how should the rubblized layer be characterized in a structural design. Thus, a large portion of this project was directed towards these two areas. Chapter 3 addresses how to assess the suitability of using rubblization given a project's site conditions, while Chapter 4 covers material characterization of the rubblized layer and other thickness design considerations.

The focus of Chapter 3 is airfields with slabs less than 9 inches and with weak underlying support. Weak support typically means little to no subbase below the PCC and a soft subgrade that often has poor drainage. The combination of thin slabs and weak underlying support can lead to problems with the rubblization process. By examining the evaluation methodology and criteria that currently exists among a few state highway agencies, a protocol and criteria is developed for airfields to assess the relative risk of not being able to successfully rubblize these "marginal support" pavements. Close examination of a few recent projects where rubblization difficulties occurred (documented in Chapter 2) provided an opportunity to develop a list of recommendations and considerations that can be utilized during the design and construction phases to maximize the probability of success.

Chapter 4 covers structural characterization of the rubblized layer and other thickness design considerations. While the FAA is moving towards designing all their airfield projects with their layer elastic method (LEDFAA), the CBR method is still predominantly used to design military airfields in the U.S. and abroad. Thus, the research team decided to address both layered elastic and CBR methodologies. Average back-calculated modulus values of rubblized layers were collected from a variety of rubblized sections and projects found in the literature. While these values were acquired from other researchers, other new values were obtained from new

analysis performed by our research team under this project. Relationships between modulus values and slab thickness are explored, along with the concept of "retained modulus," in order to provide guidance to the engineer for selecting a design modulus value of the rubblized layer. For utilizing the CBR design procedure, "equivalency factors" are suggested. In addition, Chapter 4 discusses recommended minimum hot-mix asphalt (HMA) overlay thickness criteria. Finally, a sensitivity analysis using LEDFAA is performed in Chapter 4 in order to see how the calculated design HMA overlay thicknesses varies by changing the design modulus value of the rubblized layer.

Chapter 5 reports procedures used for the quality control and assurance of the rubblization process. Comparisons are made of the current practices and criteria among agencies for performing test strips and digging test pits to ensure the desired end-product characteristics of the rubblized material are met. Recommended practices are developed for test strips and test pits, and verifiable test pit acceptance criteria is established within a performance-based specification. In addition, Chapter 5 explores future concepts of utilizing existing in-situ deflection testing or intelligent compaction technology to control rubblization quality. While these concepts offer promise, they require additional research and development before full-scale implementation.

General guidance is provided in Chapter 6 on various issues not sufficiently covered elsewhere in this report. A primary basis of the guidance in Chapter 6 is the literature review in Chapter 2.

Appendix A provides a stand-alone report of the rubblized test sections at the FAA's National Airport Pavement Test Facility (NAPTF) that were rubblized and then trafficked in 2005. HMA was placed over rubblized and unrubblized PCC with various support conditions. These sections were trafficked with simulated aircraft loadings until failure. Extensive testing was conducted by the FAA on these sections before, during and after trafficking. This NAPTF study offered a unique opportunity for extensive pavement monitoring and response testing of rubblized sections under very controlled environment and traffic loading conditions. No other research project in the literature offers such an extensive set of loading and material response data on a rubblized pavement section. Thus, Appendix A serves as a related but separate report, which describes the construction of these rubblized NAPTF sections, their loadings, the testing, and analysis that was conducted. The observations and results from NAPTF were considered heavily in formulating the guidance in this report.

In writing this report, it became essential to cross-reference the various sections and chapters in order to offer a complete discussion of the topic at hand without repeating the same information in multiple areas. The reader's patience with this is appreciated.

CHAPTER 2 – STATE OF THE PRACTICE REVIEW

After a brief introduction to the rubblization process, this chapter focuses on listing, describing and summarizing key sources of information used in this project. Because subsequent chapters pull this information together topically, this chapter is organized by source type: literature, interviews, and past projects. A more focused review of the literature pertaining to back-calculated modulus values for rubblized material is provided in Chapter 4. The chapter concludes with a discussion comparing current rubblization specifications. This chapter does not provide a complete discussion of rubblization, but rather establishes a baseline of information for the rest of this report.

2.1   Introduction to Rubblization

Rubblization is the process of fracturing the existing Portland Cement Concrete Pavement (PCCP) in-place into small interconnected pieces that serve as a base course for a new Hot Mix Asphalt (HMA) overlay. Since there are no hauling or disposal costs and none of the existing pavement system is discarded, rubblization can be a very cost-effective rehabilitation method. The rubblization process fractures the existing PCCP into small pieces, eliminating the underlying slab integrity and movements that cause reflection cracking. Any existing pavement layers below the concrete slabs, such as a crushed aggregate or stabilized base, remain in place to provide additional structural support for a new HMA pavement. Some designers have placed a dense crushed aggregate base layer, or reclaimed HMA or PCC, over the rubblized PCC as a leveling course prior to the HMA. The HMA pavement is placed as a series of lifts, whose total thickness is determined in a manner similar to how a new flexible pavement would be designed. The net effect of rubblizing is to convert a deteriorating rigid pavement system into a new flexible pavement system.

Rubblizing PCCP should result in the complete destruction of any slab action before applying the HMA overlay. For concrete pavement with distributed steel, such as jointed reinforced concrete pavements (JRCP) and continuously reinforced concrete pavement (CRCP), the steel should be ruptured or the concrete-to-steel bond should generally be broken. This is necessary to eliminate any slab action that could cause reflection cracking. Rubblization reduces the existing concrete pavement into a layer that resembles an aggregate base with a high degree of particle-to-particle interlock and is stiffer than most aggregate base layers.

2.2   Technical Literature

The relevant technical literature found and utilized for this project is listed and summarized by the following categories: *Industry Manuals*, *Published Papers*, and *Other Reports and Documents*. A bullet format is used to describe the highlights of each literature item as they most pertain to this project. The summary bullets are not intended to provide a thorough summary of each literature piece, but rather to highlight the most significant aspects and findings that most relate to this research project. Each item is referenced so the reader can probe further.

2.2.1   Industry Manuals

Asphalt Institute (AI) Manual Series (MS) No. 17, *Asphalt Overlays for Highway and Street Rehabilitation* **[AI, 1999]**
- Provides guidance for evaluating and designing asphalt overlays for both asphalt and concrete pavements.
- Nondestructive testing procedures covered in Chapter 6, which serves as a guideline in the overall pavement evaluation and design process.
- Chapter 10 covers all PCC fractured slab techniques: rubblization, crack and seat (on PCC without steel reinforcement), and break and seat (on PCC with steel reinforcement). Equipment, procedures, and design considerations are provided for each of these techniques. Chapter 10 provides design recommendations, including recommended AASHTO layer coefficient factors for the rubblized layer of between 0.16 (typical value for an unstabilized crushed stone base) and 0.30 (typical value for a stabilized base).
- Chapter 12 includes a rubblization guide specification. This was adopted essentially in-whole by the United States Air Force (USAF) in 2001 and remains their current guidance on retarding reflection cracking, published as Engineering Technical Letter (ETL) 01-09 [USAF, 2001].
- Chapter 15 covers drainage, including guidance on designing and constructing edge drains for rubblized pavements.

National Asphalt Pavement Association (NAPA) Information Series (IS) -117, *Guidelines for the Use of HMA Overlays to Rehabilitate PCC Pavements* **[NAPA, 1994]**
- Written by PCS/Law, Inc for NAPA in 1994 and covers all fractured slab technologies.
- Purpose was to present a highway design procedure, consistent with the 1993 AASHTO procedure, for determining HMA overlay thickness requirements of fractured PCC.
- Basis for the design tables and charts was a comprehensive study performed by PCS/Law in 1991 and funded by NAPA and 19 state asphalt pavement associations. This study [PCS/Law Report, 1991] included an extensive literature search and detailed field performance evaluation of 118 actual pavement sections throughout the U.S. Both condition surveys and non-destructive testing (NDT) were conducted on each section to develop performance models for the various fractured slab techniques. While all the techniques did well, rubblization had the best average performance of more than 20 years. The backcalculated modulus values found from these 118 sections are further discussed in Chapter 2 of this report.
- The design process utilizes AASHTO layer coefficients. There are three levels of evaluation and design based on a project's size and importance. As the project's size and importance increases, so does the level of engineering effort. A Level II design simply estimates a structural layer coefficient for the fractured slabs, while a Level III design calls for the use of non-destructive testing to determine project specific values.

NAPA IS-132, *Design and Construction Guidelines on Rubblizing and Overlaying PCC Pavements with HMA* **[NAPA, 2006]**
- Written by Dale Decker for NAPA and published in January 2006.
- Provides state-of-the-art guidance on the rubblization process, focusing on highways. Serves as an update and complements NAPA IS-117.

- The design process in IS-132 is based on mechanistic empirical (M-E) design principles and provides three levels of engineering evaluation. Levels I and II provide design tables and charts for PCC slab thicknesses between 7 to 12 inches, the typical range for highways. Level III calls for the use of an M-E highway design procedure such as *PerRoad* [Asphalt Pavement Alliance, 2006].
- Individual chapter topics include Rubblization Equipment, Construction Operations for Rubblization and Placing the HMA.

Federal Aviation Administration (FAA) Engineering Brief (EB) No. 66, *Rubblized Portland Cement Concrete Base Course* [FAA, 2004]
- Authored by Rodney Joel of the FAA in Feb 2004 after soliciting input from industry.
- Provides the first official and comprehensive FAA guidance, including a specification, for rubblizing PCCP.
- Allows for rubblization to be used on FAA projects, but still requires appropriate approval for a "modification to FAA Standards."
- States that once the existing PCCP has been rubblized, a new pavement surface can be constructed with either HMA or PCC.
- Section topics include: Definition, Background, Areas of Use, Advantages, Equipment, Factors Affecting Particle Size, Preparation of the Pavement Surface, Existing Structures, Drainage Systems, Relief Trenches, Test Pits, Design Procedures, Leveling Course, Traffic Control and Economic Considerations.
- Includes FAA guide specification: *Item P-215, Base Course from Rubblized Concrete Pavements.*

2.2.2 Published Papers

Transportation Research Board (TRB) Transportation Circular No. E-C087, *Rubblization of Portland Cement Concrete Pavements* [TRB, Jan 2005]
- This published compilation of six "invited" papers was presented as a two-part session at the 2005 Annual Meeting of the TRB. It offers a fairly comprehensive state-of-the art review for rubblization technology. These six paper titles, respective authors and a brief discussion are listed immediately below.

"Design and Construction of HMA Overlays on Rubblized Pavements: State of the Practice," [Decker and Hansen, 2005]
- Some overlap with NAPA's IS-132.
- The following rubblization topics are covered: evaluating existing pavement structure including distresses and soil conditions, HMA overlay thickness design, multi-head and resonant frequency breaker equipment, surface preparation, drainage, selection of equipment type, rubblization process and compaction recommendations, troubleshooting operations, quality control issues, placing the HMA overlay and performance history.
- Wisconsin's evaluation criterion is presented as an example of when it is appropriate to rubblize. They consider rubblization as a rehabilitation alternative when the PCC suffers from greater than 20% of joints needing repair, 20% of the surface needing patching or 20% of the slabs being cracked.

- Several performance studies are cited, and all conclude that rubblization performed better than any other rehabilitation method examined. States agencies are highly satisfied with rubblization as is evident by its widespread use. The only problems have been associated with weak subgrades with marginal or no bases.

"Rubblization Using Resonant Frequency Equipment," [Fitts, 2006]
- Topics include equipment and mechanics of resonant rubblization, construction procedures including edge drain installation, and problem areas.
- In 1986, New York State DOT was the first agency to use resonant rubblization. Since then, it has been successfully used on jointed plain concrete pavement (JPCP), JRCP, and CRCP ranging from city streets to interstate highways to airfields. Resonant rubblization has been used in 37 states, two Canadian provinces and several countries in Asia, Europe and South America.
- A resonant pavement breaker (RPB) is used to generate intersecting diagonal cracks through the entire PCC thickness at approximately a 45-degree angle. The rubblized concrete pieces have a high degree of angular interlock and increase in size with depth. Pieces are typically largest at the bottom of the slab or below any reinforcing steel.
- Problems that have occurred were generally on projects where the pavement structure was too weak to be effectively rubblized. This is typically a combination of relatively thin PCC slabs over a weak, often saturated subgrade. The resulting problems are severe rutting by the RPB tires (two of which operate on the broken PCC) and/or large blocks of concrete getting reoriented and pushed into the soft foundation. Utilizing wide flotation tires on the RPB have helped reduce this problem, but it still can occur. Installation of edge drains prior to rubblization is important to minimize this concern. On some occasions, these problems were temporary, during only the spring thaw period or after a long stretch of rain, and went away once the foundation dried.

"Rubblization Using Multi-head Breaker Equipment," [Thompson, 2006]
- Topics include development of the multi-head breaker (MHB), production capabilities, distinctive features, typical construction sequence, representative prices and performance data.
- In 1995, a MHB was first utilized to rubblize 6 to 9 inches of PCC in Wisconsin and other neighboring states. In 1998, a 14-inch thick PCC Interstate highway in Illinois was effectively rubblized by modifying the MHB equipment to increase the breaking energy. For slabs greater than 14 inches, a guillotine-style pavement breaker is typically utilized in front of the MHB to pre-fracture the PCC full depth.
- The MHB operates as a single pass machine with a breaking width of one vehicle-lane. An advantage of the MHB on highways is that rolling and HMA lay-down operations can take place directly behind the rubblization operation. The operating characteristics (hammer weights, number of hammers, hammer drop heights and travel speed) can be varied to produce a wide range of fracture energies to accommodate project conditions.

"Rubblization of Airfield Pavements - State of the Practice," [Buncher and Jones, 2006]
- Focus is on airfields versus highways.

- Topics include process, equipment, specification development for airfields, listing of airfield projects over the past 7 years that have used rubblization, listing of upcoming airfield rubblization projects, and design and construction issues.
- At Wright-Patterson Air Force Base, Ohio, a JPCP parking apron between 21 and 26 inches thick (no steel) was successfully rubblized with a RPB in 2002. An RB-500 by Resonant Machines, Inc (RMI) was used. Even though the rubblization was to facilitate removal, the JPCP was thoroughly rubblized and broken into pieces no larger than 12 inches. The 63,000 yd^2 apron was rubblized in 10 days.
- At Selfridge Air National Guard Base, Michigan, a JPCP runway up to 21 inches thick was successfully rubblized with a MHB in 2002. The concrete was first broken using a guillotine-type breaker and then rubblized with the MHB. A test pit confirmed specification compliance before full production began. The 95,000 yd^2 runway was rubblized in 16 days. An extensive underdrain system was installed in advance of the rubblization process. Recycled PCC from both touchdown ends was processed through a portable crusher and placed as a 4-inch leveling course, before the 7 inch HMA overlay.

"Nondestructive Testing Results from Rubblized Concrete Pavement on Interstate 10 in Louisiana," [Scullion, 2006]
- Texas' limited experience with rubblization has not been good because of selecting projects with marginal conditions (thin slabs, no base and soft subgrade). This evaluation was to learn what Louisiana, which has had excellent results with rubblization, is doing differently and if those practices can be successfully applied to Texas.
- A summary of ground penetrating radar (GPR) and falling weight deflectometer (FWD) results are presented from rubblized sections 2 to 3 years old on I-10 in Louisiana.
- The sections are performing very well. The rubblized layer has a modulus 10 times greater than that of a traditional flexible base. There is no evidence of trapped water in the base. Installation of edge drains, standard practice in Louisiana, is critical.
- Rubblization is preferred in Louisiana over crack and seat when failed and voided joints are present because crack and seat may simply bridge the voided joints while the rubblization and rolling process tends to smooth out the profile.
- Rubblization in Louisiana is aided by the fact that most their interstates have stiff cement-treated bases beneath old JRCP, while Texas built their PCC directly over select fill.

"Illinois Performance Study of Pavement Rubblization," [Wienrank and Lippert, 2006]
- Provides a history of rubblization in Illinois. There have been 12 experimental rubblization projects on state routes in Illinois since 1990 utilizing either the MHB or the RPB, eight of which have been monitored for performance with IRI ratings. Design details of these eight projects and a summary of their field performance are provided.
- HMA overlay thicknesses were varied between sections on most of these projects, which included rubblized sections and non-rubblized sections (control). To date, the rubblized sections on all eight of these projects have performed equal to or better than the control.
- On one project, the "degree" of rubblization was varied to determine if there were any negative effects to a more rapid rubblization procedure (larger particle sizes). There has been no noticeable difference in performance between the different sections.

- PCC pavements that have severe joint distresses and transverse cracking or extensive punchouts (in CRCP) are good candidates for rubblization. In addition, PCC pavements that have durability cracking (D-cracking) are also ideal candidates.
- A thorough investigation of subgrade soils is suggested when considering rubblization to determine if ample support is available for effective rubblization. The rubblized pavement, subbase and subgrade must be capable of supporting construction equipment. In Illinois, the MHB is preferred on projects where there are marginal support conditions expected because the MHB only operates on unbroken PCC, whereas half the RPB operates on the rubblized PCC.

2.2.3 Other Reports and Documents

***Performance of Colorado's First Rubblization Project on I-76 Near Sterling,* [LaForce for CDOT, 2006]**
- While a separate CDOT report in 2000 documents the design, construction and post-construction evaluation of this project [CDOT, 2000], this report summarizes that information, analyzes the six-year performance of the sections, and performs a life cycle cost comparison of the project versus an un-bonded concrete overlay alternative.
- Both the MHB and RPB were used on separate sections. A crack and seat section was also planned, but the guillotine hammer was unable to fracture the JPCP because of the severe alkali-silica reactivity (ASR) damage that existed within the PCC on the entire project. Edge drains were installed prior to rubblization. Six inches of HMA was placed over the rubblized sections.
- Field performance has been outstanding, with no reflection cracking, rutting, settlement or other distresses related to the rubblization process.
- Both the MHB and RPB appeared to accomplish the required fracture and will be allowed on future projects. Even though the concrete had severe ASR damage which resulted in tightly locked-up slabs, no special requirements were needed to successfully rubblize with either rubblization method. In addition, there has been no performance issues related to the ASR.
- The edge drains were effective in preventing moisture from accumulating under the pavement and will be used with future CDOT rubblization, unless the subgrade is free draining.
- The rubblization alternative (including edge drain installation and maintenance) had a lower initial cost and a lower 40-year life cycle cost versus the unbonded PCC overlay alternative. For future life cycle cost analysis, CDOT will treat a rubblized pavement the same as a new HMA pavement in terms of rehabilitation and maintenance.

***Performance of Rubblized Pavement Sections in Alabama*, [Timm and Warren for AL DOT, 2004]**
- The objective of this study was to assess the overall performance of 11 rubblized sections that were constructed between 1991 and 2001 in Alabama.
- The PCC thicknesses on these 11 sections were between 8 and 10 inches, with two being CRCP and the others being JPCP or JRCP. Edge drains were installed on all but two of these sections, and those two already had drainable bases. The HMA overlay thicknesses were between 7" and 12".

- Performance evaluations looked at rutting, cracking and IRI. Rutting was minimal, transverse crack spacing was greater than 80 feet, and the IRI was very good on all 11 sections.
- Higher levels of distress were present on rubblized CRCP (also the only two that had a stabilized base) and on thicker PCC. While this was not investigated further, the report suggests that close monitoring should occur during rubblization on these types of pavements to ensure the steel is debonded from the concrete and there is adequate fracture within the PCC.
- Since the average age of these projects was 5 years and the average HMA overlay thickness was 10.5 inches, it is possible that sections which may not have been effectively rubblized could still develop reflection cracking in the future.
- In summary, the report concludes that rubblization has been an effective and efficient rehabilitation alternative in AL. The sections have met or exceeded expected performance criteria.

***Rubblizing with Bituminous Concrete Overlay – 10 Years' Experience in Illinois*, [Heckel for IL DOT, 2002]**
- This report overlaps considerably with the paper by Weinrank et al in 2005. As of 2000, ten IL DOT projects had been constructed utilizing rubblization (the first in 1990) and none have needed rehabilitation. Seven of these projects had experimental features and have been closely monitored. Lessons learned from these seven are provided.
- On IL 38, the minimum HMA thickness to prevent damage to the rubblized pavement by traffic was determined to be 4 inches. IL DOT design guidelines state the minimum overlay thickness is 6 inches, with the first lift being 3 to 4 inches to achieve adequate compaction and to minimize damage to the rubblized material. Surface lifts should be at least 1.5 to 2 inches thick.
- On I-57, the use of a tracked paver disturbed the rubblized layer much less than a rubber-tired paver.
- Rubblization has proven to be a successful rehabilitation method on both Interstates and non-Interstate projects. Reflection cracking has been minimal. Performance has been very good, with less reflection cracking than on adjacent "patch and overlay" sections.

***Identify Causes for Under Performing Rubblized Concrete Pavement Projects,* [Baladi and Svasdisant for MI DOT, 2002]**
- Michigan DOT has constructed 86 rubblization projects since 1988. While some have performed well and are expected to meet their design life of 20 years, others have underperformed. The objective of this study was to determine the causes of under performance and recommend solutions.
- Fifteen rubblized pavements were thoroughly investigated during the construction phases as well as after opening them to traffic. This consisted of excavating trenches, non-destructive deflection testing and coring, lab testing, distress surveys, and finite element analysis. The MHB was used on nine projects while the RPB was used on the other six. The range of HMA overlay thicknesses was 5 to 8 inches.
- The major distresses contributing to the underperformance were raveling, joint reflection cracking and top-down cracking (TDC). While raveling is attributed to poor HMA quality and not rubblization, reflection cracking may be related to poor rubblization

quality if the PCC slab integrity was not destroyed. TDC initiates at the HMA surface and propagates down and out over time, caused by excessive tensile stresses at the top of the HMA. The authors hypothesize that TDC is also related to poor rubblization quality if the stiffness of the rubblized material is highly variable. A 3-D mechanistic analysis showed high load-induced tensile stresses at the HMA surface when a sandwich model exists from a stiff HMA layer over a softer "rubblized" PCC layer over a much stiffer "fractured" PCC layer. While Baladi refers to the top of the rubblized layer as "rubblized", we will refer to it in this report as "crushed" for less confusion.

- Trenches were excavated on the 15 projects to determine if specification requirements were met; namely, particle size less than 6 inches, debonding of the temperature steel, and destroying of the PCC joints. The following observations were made:

   1. Both rubblization methods (RPB and MHB) produced two different layers in the PCC as depicted in Figure 2.1; a "crushed" ("rubblized") layer at the top and a "fractured" layer at the bottom. Figure 2.1 illustrates that the thickness of these two layers varies from one location to another. Some locations had the "fractured" PCC extend to the surface while others had the "crushed" material extend to the bottom of the slab. The author estimated that the stiffness (modulus) of the "fractured" PCC was an order of magnitude higher than the "crushed" material. This is further examined in Chapter 4. The "fractured" layer was generally below the temperature steel and the "crushed" layer generally above, but not always.
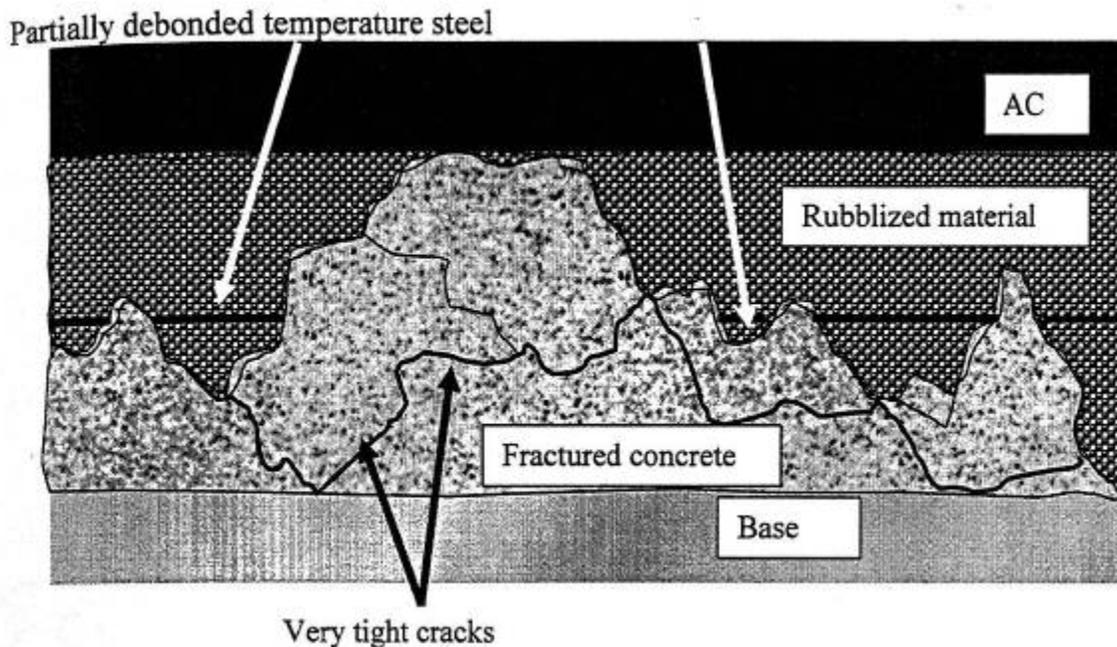
Figure 2.1, Schematic of "Crushed" and "Fractured" Layers [Baladi et al, 2002]

   2. The RPB was successful in debonding the temperature steel from the PCC in most places, except where the steel overlapped and the two steel layers hindered the rubblization process by absorbing and/or deflecting part of the energy. The MHB was generally not successful, however, as only 15% of the temperature steel was

debonded from the PCC, as shown in Figure 2.2. Where the temperature steel overlapped, the MHB did not debond the steel.

Figure 2.2, Photo Showing Partial Debonding of Temperature Steel [Baladi et al, 2002]

3. For both rubblization methods, trenches made along longitudinal and transverse joints revealed significantly larger particles (about twice the size) compared to those examined along trenches made at mid-slab. The dowel bars at the joints possibly absorbed and deflected some fracture energy. In some cases the integrity of the joint was not likely destroyed as dowel bars were embedded in the PCC on both sides of the joint, as shown in Figure 2.3.
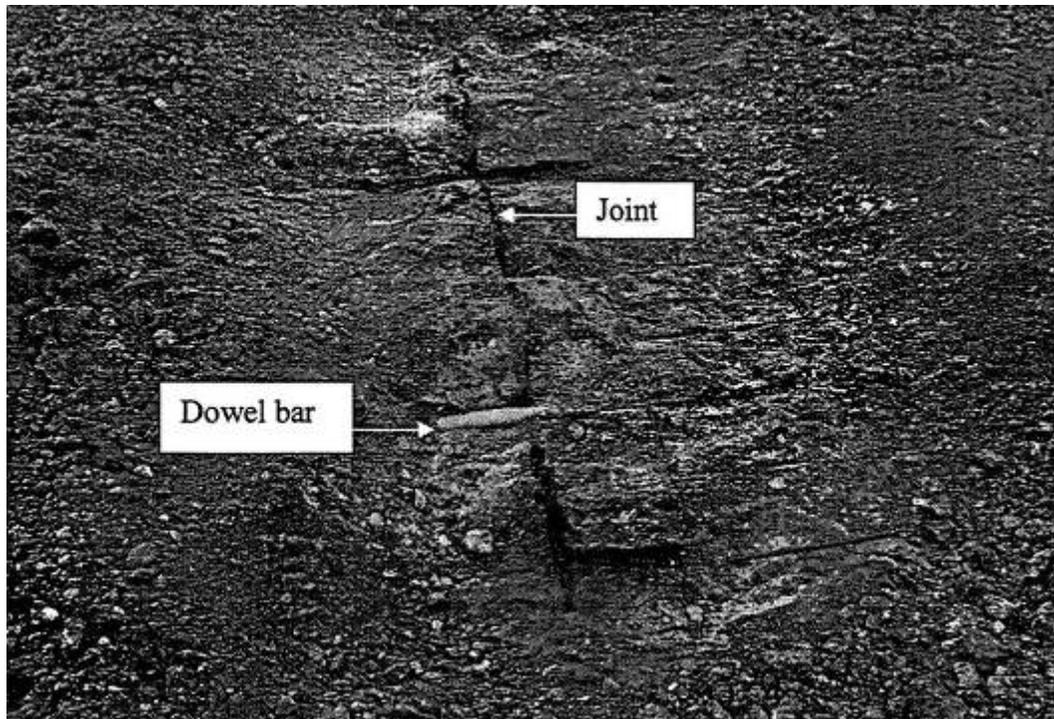
Figure 2.3, Photo Showing Only Partial Destruction of Joint Integrity [Baladi et al, 2002]

4. For both rubblization methods, the permeability of the fractured PCC was at times poor, causing concern by the authors that any water infiltrating through the HMA overlay could be trapped, increasing the potential of stripping in the HMA.
5. For the MHB in several areas, some large (> 6 inches) PCC pieces were rotated and penetrating into the subbase or subgrade to a depth of one inch. The author suggests that better calibration of the MHB equipment (speed and hammer drop height) could prevent this "localized shear failure."
6. On most of the RPB projects, longitudinal strips of fine and course particles were evident, spaced at the same width as the resonant breaking shoe. While this non-uniformity of the rubblized layer is mentioned, no further cause of concern is cited. The author also mentions that the surface of rubblized slabs is generally raised 1 inch relative to that of un-rubblized slabs.

- Listed below are some of the report's major conclusions and recommendations:
  1. Rubblization is a viable rehab alternative that requires more detailed quality control measures than conventional asphalt pavements due to the TDC issue.
  2. The RPB generally delivered a better and more uniform rubblized layer compared to the MHB. Specifically, the RPB delivered a higher percentage of debonded steel, a thicker and more uniform depth of rubblized material and better destruction of joint integrity.
  3. Concrete pavements that are poor candidates for rubblization include those that either have been crack and seated, have extensive structural cracking, have a soft subbase (modulus < 7000 psi) or a soft subgrade (modulus < 3000 psi).

4. The rubblization process should not commence during periods when the subgrade is likely to be saturated, such as during the Spring season thaw or after a heavy rain.

Evaluation of Rubblized Pavement Sections in Michigan Constructed between 1988 - 2002, Applied Pavement Technology Inc. (APT) for Antigo Const. Inc., [APT, 2006]

- Antigo requested APT to review the performance of the rubblized pavement sections reported in the 2002 Baladi report, as well as determine if there was a relationship between performance and the level of rubblized uniformity found in the trenches by Baladi. This study was also documented in a 2007 TRB paper [Wolters, 2007].
- Using MI DOT's own Distress Index (DI), Ride Quality Index (RQI) and International Roughness Index (IRI) measurements, almost all rubblized sections constructed after 1997 are performing very well. Specifically, 20 of 21 MHB sections constructed between 1997 and 2002 had a DI, RQI and IRI rating of good or better. All 36 RPB sections constructed during the same timeframe had a DI, RQI and IRI rating of good or better.
- Prior to 1997, only the RPB was used. About a third of these older rubblized sections had DI ratings of fair or less, but the distresses were not predominantly reflective cracking.
- For the majority of MHB and RPB sections, the primary distress is longitudinal cracking at the longitudinal joint or along the pavement edge, which is not attributed to rubblization.
- Using the DI as a method of estimating service life, various subsets of MHB and RPB data were analyzed. There was no significant difference between the two methods. Combining the MHB and RPB data sets revealed an estimated service life of 16.5 years.
- The authors separated those sections constructed as "partially drained" from those that were "fully drained" because the "partially drained" sections were clearly in worse condition. The expected service life of "fully drained" sections was 21 years, while expected service life was only 9 years for "partially drained" sections.
- A qualitative comparison was made of the uniformity results from the trenches in the 2002 Baladi report. There was poor correlation between how the material rubblized (based on uniformity of particle gradation) and the current pavement condition.
- Distresses on the 21 MHB sections and 36 RPB sections are mostly from HMA segregation and poor paving joints, not related to rubblization.

***Summary and Evaluation of Rubblized Pavement Test Results at FAA's NAPTF*, Appendix A of this report, written by McQueen**

- A complete discussion of the rubblized sections that were constructed, loaded and tested in 2005 at the FAA's National Airport Pavement Test Facility (NAPTF) is located in Appendix A of this report. Roy McQueen of McQueen and Associates wrote Appendix A as a stand-alone report. Roy's efforts were an integral part of this AAPTP 04-01 project.
- The primary purpose of the NAPTF is to generate full-scale pavement response and performance data for the development/verification of airport pavement thickness design procedures and criteria. The discussion here is limited to a brief discussion of the test sections, trafficking and observations. Chapter 4 provides a summary of the results and analysis pertaining to structural characterization of the rubblized layer.

- The three distinct test sections are designated MRC, MRG and MRS as shown in Figure 2.4. The northern half from centerline was rubblized while the southern half was not. All sections consisted of 12 inches of JPCP (no reinforcing steel but with doweled joints). All were built over a medium strength subgrade with a CBR range of 7-8. All the rubblization occurred with a RPB (RMI's PB-500). All sections were overlaid with 5 inches of HMA (P-401 material). MR**C** had a 10-inch **conventional** crushed stone subbase (P-154), MR**G** had slabs directly on **grade**, and MR**S** had a 6-inch **stabilized** base (econocrete, P-306) over a 6-inch subbase (sandy gravel, P-154).
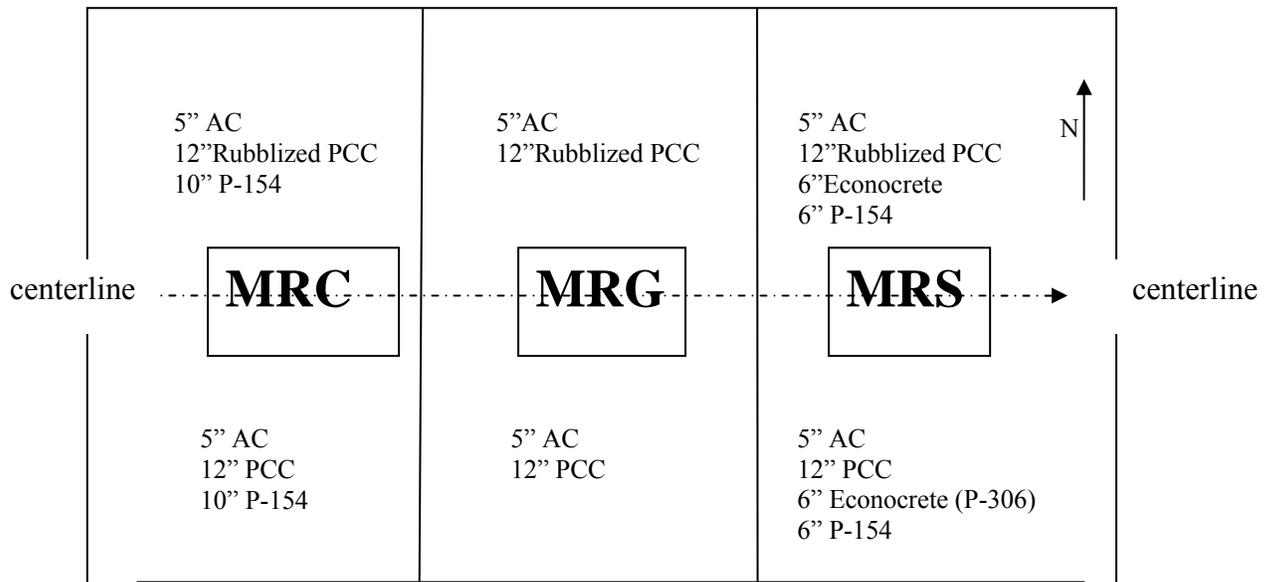
Figure 2.4, Layout of Test Sections at NAPTF [Garg, 2006]

- All pavement layers were tested during construction. After rubblization, test pits were excavated to observe the rubblized material and to access the subgrade and base for testing. Test pits indicated the top 2 to 3 inches of rubblized PCC was broken down into particles smaller than one inch, while the particles in the bottom 9 inches of rubblized PCC had dimensions as large as 12 to 24 inches.
- After the HMA was placed, trafficking began with a dual tandem (4-wheel) configuration of 55,000 lbs. This loading was selected to bring the sections to failure. However, after over 5,000 repetitions, less than ¼-inch rutting was observed on all the sections. The FAA therefore decided to increase the loading to 65,000 lbs with a triple tandem (6-wheel) configuration. Significant rutting then developed and failure was declared on the MRC sections just after 15,000 repetitions. The MRG and MRS sections were loaded through 25,000 repetitions before terminating. Profile measurements plotted versus load repetitions are in Appendix A.
- Post traffic trenching revealed that subgrade shear failure occurred in MRC as shown in Figure 2.5 and 2.6. The P-154 subbase (dark layer shown in Figures 2.5 and 2.6) only deformed as the subgrade deformed. Note the subgrade intrusion into the P-154 subbase in Figure 2.6. Unexpectedly, the subgrade in MRC sheared before the MRG subgrade. It

is theorized that this was due to the MRC subgrade being unintentionally weakened from moisture draining down off the P-154 material when it was placed. CBR tests taken from inside the trenches support this, where the MRC and MRS sections had subgrade CBRs averaging 4 while the MRG sections had subgrade CBRs averaging 11.

- Regarding the rubblization process, the test pits showed there were very large PCC pieces at the bottom of the slab, and some of the dowel bars were still embedded in the PCC after rubblization. This is further discussed in Chapter 4. Fracture was full depth.
- On the MRS section, the econocrete base did not appear damaged or cracked from the rubblization process.
- The work at NAPTF on these rubblized sections is also described in a paper presented at an FAA Worldwide Conference [Garg and Hayhoe, 2007].

Figure 2.5, Trench in MRC Showing Subgrade Shear Failure [Garg, 2006]

Figure 2.6, Close-up in MRC Trench Showing Subgrade Shear Failure (Garg, 2006)

2.3   Interviews and Discussions

Interviews were conducted with many rubblization experts to discuss the many issues and questions that relate to this research project. These experts included John Anderson (Tigerbrain Engineering Inc.), Phil Kirk (Resonant Machines Inc.), George Shinners and Matt Shinners (Antigo Construction Inc), Jay Hensley (Private Consultant), Vern Barnhart (Michigan DOT), Chris Abadie (Louisiana DOT) and Irv Dukatz (Mathy Construction). Their vast experience and opinions helped immensely with shaping the recommendations provided in this report.

2.4   Airfield Projects

Tables 2.1 and 2.2 list past airfield rubblization projects reported to have been accomplished by Antigo Construction Inc (operates the MHB) and Resonant Machines Inc (operates the RPB), respectively. The total is about 30 projects which approximate 1.5 million square yards (SYs). We are not aware of other airfield rubblization projects other than those listed. Both tables include the airfield/location, year, SYs and thickness of PCC rubblized. Antigo also provided the thickness of the HMA overlay and some comments, which are part of Table 2.1. The comments in Table 2.1 include when the guillotine-type breaker was used to pre-break the PCC (typically when PCC is more than 14 inches) and when an unbound leveling course was utilized prior to the overlay. Both tables include projects where an HMA overlay was not utilized, either because a PCC overlay was placed or the rubblization was strictly for removal of the PCC. By listing the projects chronologically, one can see the significance of the Wright Patterson and Selfridge projects because those were the first demonstration of rubblization on heavy-load pavements with PCC thicknesses significantly greater than highways [Buncher, 2006].

Additional statistics were provided by Antigo. Total SYs of crack/break & seat performed by Antigo between 1982 and 2006 was 62.9 million, while 1.5 million of that total was on airfields (2.4%). Total SYs of rubblization performed by Antigo between 1982 and 2006 was 25.2 million, while 0.7 million of that total was on airfields (2.8%). The ratio of total crack/break and seat versus total rubblization by Antigo is 2.5 to 1, while the same type of ratio for strictly airfields by Antigo was 2.1 to 1. With Antigo rubblizing 25.2 million SYs of PCCP, and the total amount of rubblized PCCP being 50 million SYs, it appears the market share for all rubblized pavement is split fairly evenly between the MHB and RPB.

Installation, Location, Airfield Designation	Year	Concrete Rubblized (sy)	Thickness of Existing Concrete (inches)	HMA Overlay Thickness (inches)	Comments
Rantoul Airport, IL, R/W 18-36 & T/W 'F'	1999	45,670	6, 8	5	Demonstrated 3 maximum surface particle sizes: 3", 9", 18"
Columbus Airport, IN, T/W 'C' & West Apron Taxilane	2000	24,975	6	7.5	
Watertown Airport, SD, Apron Area	2001	39,456	8	4	Asphalt millings (3" thick) placed prior to HMA overlay
Rantoul Airport, IL, R/W 9-27	2001	3,625	6-8	4.6	
Kalamazoo/Battle Creek Airport, MI, T/W 'E'	2002	5,250	8	11.5	
Selfridge ANG Base, Detroit, MI, R/W 01-19	2002	95,706	13, 16, 19 and 21	7	Pre-break w/ guillotine, crushed agg. (4.5" th) for grade correction
IN ANG Base, Ft. Wayne, IN, Parking Ramp & T/W	2003	25,258	10	-	13" JPCP overlay
Watertown Airport, SD, Hanger area	2003	1,982	6	3	Asphalt millings (3" thick) placed prior to HMA overlay
Ephrata Airport, WA, R/W 11-29 & T/W B-2	2004	26,500	6	4	
Columbus Airport, IN, T/W 'E'	2004	12,768	6	7	
Capital Airport, Springfield, IL, R/W 4 O.R.	2005	15,000	10	3	
Grand Forks AF Base, ND, R/W 17-35	2005	237,558	19-23	9	Pre-break w/ guillotine, crushed agg. (4 - 13" thick) for new crown
Buffalo Niagara Airport, NY, T/W 'A'	2005-06	21,562	11	8	
San Juan International A.P, Puerto Rico, R/W 10-28	2005-07	147,143	15	-	Pre-break w/ guillotine, crushed agg. (3-6") prior to PCC overlay
Pierre Airport, SD, R/W 13-31 Blast Pad	2005	4,984	9	4	
Toledo Metcalf Field, OH, R/W 4-22	2006	29,542	6	3	Crushed aggregate (variable) placed prior to HMA overlay
Detroit Metro Airport, MI, Connector W-1	2006	625	17	5-9	Feasibility test for Detroit Metro runways & taxiways

Table 2.1, Antigo's (MHB) Airfield Rubblization Projects from 1999-2006 [Antigo, 2007]

Installation, Location	Year	Concrete Rubblized (sy)	Thickness of Existing Concrete (inches)
Navy Air Base, C-17 Assault Strip, SC	1995	50,000	9
Jacksonville Navy Air Station, TW A, FL	1997	26,000	11
Grant County Airport, RW 4-22, Moses Lake, WA	2000	55,000 of which 31,000 removed	6
Wright Patterson Air Force Base, Parking Apron, Dayton, OH	2002	65,000 All for Removal	21-26
Walla Walla Regional Airport, RW, Walla Walla, WA	2003	140,000	7-9
Buffalo/Niagara Airport, TW A, Buffalo, NY	2003	50,000	12
	2004	36,500	12
Air National Guard Base, Rochester, NY	2004	20,000	12
Naval Air Station, NJ	2004	6,000	9-12
	2005	6,000	9-12
Pratt Airport, R/W 17-35, Pratt, KS	2005	62,400 PCC Overlay	7-8
Dover Air Force Base, Dover, DE	2005	80,000	10-20
Hanscomb Air Force Base	2005	12,500	10-12
Kegelman Auxiliary Field, R/W, near Vance Air Force Base, Enid, OK	2006	135,000	6
Martinsburg Air National Guard Base, WV	2007	37,000	8, 10

Table 2.2, RMI's (RPB) Airfield Rubblization Projects (RMI, 2007)

Some of the rubblization projects listed in Tables 2.1 and 2.2 were examined closely as part of this research effort. Below is a listing of those rubblization projects, along with available references and a few bullets describing the significance of the project as related to this research effort. Project plans and specifications were also acquired in some cases as part of this effort, but are not included in this report.

Selfridge ANG Base Runway, [Anderson, 2004], [Velez-Vega, 2005]
- Anderson describes the background and details of this project, which was large in scope (runway interior) and had 21-inch thick PCC slabs that were rubblized. The guillotine hammer was utilized to pre-break the PCC prior to the MHB.
- Edge drains were installed. The rubblization process and rest of the construction went well. The 5-year performance has been good.
- Velez-Vega examined the use of rubblization on several military projects, including this project.

Rantoul Airport Runway Study [Buttlar, 2000]
- As reported by Buttlar, this study included the construction, instrumentation and monitoring of multiple test sections utilizing various crack relief techniques. All the rubblized test sections utilized the MHB, and its operation varied between test sections to attempt to produce three different gradations: 3-inch max, 9-inch max and 18-inch max at the surface. Besides these three distinct rubblization techniques, test sections also included a "saw-and-seal" technique and an "interlayer stress absorbing composite" technique.
- All the different crack relief techniques had paired sections, with one side of the centerline utilizing a conventional asphalt binder while the other side utilizing a modified asphalt binder in the 5-inch HMA overlay.
- PCC slab thicknesses were either 6-inches or 8-inches, directly on a subgrade with a CBR ranging from 2-7 predominantly, but with isolated areas as high as 17.
- Verbal reports of visual inspections as of this writing state that there is little performance difference between these test sections, with no reflective cracking through the 5-inch HMA overlay after 8 years, except for a few spots where the largest rubblized pieces may be "floating." There has been very little traffic on any of these sections.

Grand Forks AFB Runway [Velez-Vega, 2005], [Wierenga, 2006]
- This 2005 airfield rubblization project is clearly the largest to date in terms of cost ($27.5M) and rubblized area (237,000 sy). The PCC slabs were predominantly 19 inches thick (some thicker), and consisted of a very hard river-run gravel aggregate. A 15-inch gravel base and 34-inch sand subbase were below the slabs for frost protection.
- Two RB-500 units (RPBs) were first utilized on the project and test pits confirmed that specification gradation requirements were met. However, the RPBs had multiple breakdowns and the prime contractor decided production was too slow, so they were replaced early in the project with multiple guillotine hammers and MHBs to complete the project. Test pits of the MHB showed PCC gradation requirements were met at the surface, but some pieces at the bottom of the slab in the thickest sections (24 inches-thick) were as large as 36 inches in length. The engineer/owner, who was experienced with rubblization, determined that the rubblization process was adequate in these thicker sections.
- The PCC that was removed from both runway ends was run through a crusher and re-used as a base course above the rubblized surface to correct grade for a new offset crown.
- The project also included re-installing a substantial underdrain system along the centerline and both edges of the new runway, which required trenching through the full

depth of PCC. These trenches were also utilized for new runway lighting electrical conduit.

Grant County Airport Runway [Rue, 2001]
- Rue describes the background, design and construction of this project, which rubblized half (5000 ft) of a 100 ft wide runway. The 6-inch thick slabs were non-reinforced, thickened edged and rested above a marginal quality pit-run gravel base that was 7 to 15 inches thick. The subgrade design CBR was 7. No edge drains were installed because the subgrade was believed to be self-draining. A 9-inch HMA overlay was placed.
- The RPB was utilized and reported to work well. Of the 55,000 sys rubblized, 24,000 sys were designed to remain in-place and be overlaid. In the rubblized sections, there were rubble pieces larger than the specifications allowed (maximum of 8-inches). In some areas, the occurrence of these larger than specified pieces was primarily located along the joints; while in other areas, the larger pieces occurred throughout the slabs. Since the RPB requires a certain level of subgrade resistance to work (putting slab in tension), a subgrade that is wetter (and softer) at the joints can cause less effective rubblization. In a 100 by 200-ft area, a high concentration of these larger pieces occurred and the RPB equipment left tire ruts in the rubble and even pumped the subgrade to the surface. A full-depth repair was made by removing the rubble, base and subgrade and then replacing with crusher run gravel and topping with 6 inches of P-209 crushed aggregate.

Pratt Airport Runway
- Information on this 2005 project was obtained by talking to the project engineer, Dave Hadel (Burns and McDonnell, Inc.) the prime contractor, Alan Farrington (Klaver Construction, Inc.) and the rubblization contractor, Phil Kirk (RMI, Inc.). Photos and a PowerPoint presentation was also obtained from Dave Hadel.
- Specifications called for using a RPB exclusively (not a MHB) to rubblize a 100-ft wide by 5500-ft long section. A leveling course averaging about 3-inches was placed with a spreader over the rubblized surface using either HMA millings or crusher run concrete material utilizing an on-site crusher. A 7-inch PCC overlay (white-topping) was then placed.
- Existing JPCP was 7 to 8 inches thick with virtually no base underneath. The clay subgrade had soaked lab CBRs of 1.9 to 3.5.
- A 4 to 9-inch thick HMA overlay had to be milled off, and edge drains were installed full length on both sides of the runway, prior to rubblization. No water was reported to have drained, even after rubblization.
- The rubblization process started with the test section at the outside of the runway and things went well. However, as the rubblization operation moved toward the centerline, larger pieces were observed and stability problems started occurring, especially at the bottom of a swale in the runway.
- These stability problems equated to large unbroken PCC pieces (3-ft) on the surface (Figure 2.7), as well as severe rutting and several "punch-thrus" from the RPB model PB-3 (Figure 2.8). All of the PCC and subgrade "muck" in these unstable areas was removed and replaced with crusher-run material. The largest of these full depth "patch" area was 100-ft wide by 700-ft long. All the patches were large enough to allow the rollers access

to prepare the subgrade (Figure 2.9 and Figure 2.10). <u>These full depth patches accounted for about 45% of the total area rubblized by the RPB on the first half of this project.</u>

- The second half of the project utilized an impact roller device called *Impactor 2000* to break the concrete. This tow-behind device was developed by its manufacturer for demolishing the PCC <u>exclusively for removal</u>, and not for leaving the broken PCC in place, because the large pieces leave a very uneven surface. The project engineer reported that was the case on this project, with the surface being very rough and having large (24-inch) pieces. Some full depth patching was also necessary in this second half of the project. This rough surface would not have been adequate for a HMA overlay.

Figure 2.7, Ineffective Rubblization (Large PCC Pieces) at Pratt Due to Weak Support

Figure 2.8, RPB with High Flotation Tires Stuck in Unstable Area at Pratt

Figure 2.9, Excavation of Ineffective Rubblized Areas at Pratt for Full-depth Patch

Figure 2.10, Finished Patched Areas Outlined in White at Pratt using Crusher Run

Kegelman Auxillary Field Runway

- Information on this 2006 project came from the USAF Major Command Engineer (Everitt Dodson), local Project Engineer (Jason Brinkley) and the RPB contractor (RMI, Phil Kirk) . This was another project with "marginal" conditions for effective rubblization. Based on previous USAF Airfield Pavement Evaluation Reports, the plain PCCP slabs were 4.7 to 6.5 inches thick with either a thin or no sand subbase over a silty-sand subgrade that "turned to mush when wet." The subgrade K-value from a USAF evaluation test pit was 108. Drainage was poor.
- A 5-inch HMA overlay was placed directly over the rubblized surface.
- No drainage system was installed due to budget issues. The specification called for using the RPB exclusively. Rubblization started in the Spring under normal wet conditions.
- The RPB did not punch-thru, but excessive pumping and rutting did occurred (Figure 2.11). RMI stated that if rutting did not exceed 2-inches, the rollers could remove those ruts without detrimental effects. High-float tires on the RPB were not used at the start, but were later and helped reduce rutting.
- By the end of the project, the percent of full-depth patching was about 30% of the rubblized area. This percentage grew as the process moved towards the centerline. In the unstable areas, the engineers decided to excavate 2 to 4-ft into the subgrade to ensure all the "mush" was removed (Figure 2.12). Crushed PCC and aggregate was used as patch material. The total overrun cost of these excavated and patched areas was about $1.2M.

Figure 2.11, Rutting from RPB of Unstable Area at Kegelman that Requires Excavation

Figure 2.12, Full-depth Excavation Deep Into Subgrade of Patched Area at Kegelman

Tullahoma Regional Airport Runway Feasibility Trial
- This closed secondary runway 06/24 was structurally evaluated for rehabilitation in December 2006 [Applied Research Associates, 2007]. The ARA report includes results of the pavement condition inspection, and FWD, coring, DCP, and soil boring testing. The report also includes six preliminary design alternatives for the design aircraft (400 annual departures of a 60,000 lb aircraft and 3,000 annual departures of a 10,000 lb aircraft). Of the six, the most cost effective rehabilitation alternative was rubblization with an asphalt overlay. Two HMA overlay designs were offered; 1) 7.5-inch HMA over rubblized and 2) 4-inch HMA over 4-inch crushed aggregate over rubblized.
- The entire runway had a slab thickness of 7 inches, no steel, no base or subbase, and variable subgrade strength. The subgrade was a sandy clay with a moisture content generally higher than the plastic limit. While the values selected for design were a CBR of 4 and modulus of 112 pci, the DCP testing showed subgrade CBR values ranging from 1 to 14 with no logical pattern in the DCP and FWD results showing any sections that were consistently stronger or weaker than others.
- The FAA referred the project consultant (PDC Consultants) to our research team in April 2007 for suggestions on designing and executing the Tullahoma project due to our involvement with this research. After considerable discussions, we were successful in convincing PDC Consultants of the importance to schedule a feasibility trial on the closed runway where both types of rubblization equipment (MHB and RPB) could be brought in to demonstrate whether rubblization with their equipment was feasible, and if one type would be better suited than the other. Both RMI and Antigo were very cooperative in participating. We attended the feasibility trial in May 2007 as part of this project to observe and provide advice.

- The results of the feasibility trial are covered in a report [Hall, 2007], and also in Section 3.3 of this report. In general, the thin PCC and soft subgrade did not provide adequate support for either machine to rubblize the pavement and meet typical particle size criteria found in a specification. Achieving full-depth fracture was not a problem with either type of equipment, but the RPB did have problems in some wetter areas by rutting the rubblized surface. The MHB was able to lower its drop height and space out its impacts in order to provide a surface that appeared to be acceptable for an overlay (Figure 2.13). However, a big concern was whether the fractured pavement would be able to support ensuing construction traffic. Observations and alternatives are discussed more thoroughly in Section 3.3 on this report.

Figure 2.13, Fractured PCC by MHB with Low Drop Ht. and Wide Spacing at Tullahoma

- After further consultation with us through PDC, Hall's report recommends that this project be changed to utilize the crack and seat method of rehabilitation (described in Section 3.3 on this report). The HMA overlay thickness requirement calculated by Hall is 7-inches, directly over crack and seated PCC. The project is scheduled to start in Summer, 2008.

Detroit Metropolitan Airport Demonstration Sections [Decker, 2006], [Kohn, 2006]
- Two short test sections at the Detroit Metropolitan Airport were rubblized and overlaid in 2006 to determine the applicability of using either the RPB or MHB equipment for future runway and taxiway rehabilitation projects. Both the RPB section and the MHB section consisted of 17-inch thick PCC that had significant alkali-silica reaction (ASR) damage

and wire reinforcement (4 inches below surface) on top of 8 inches of HMA base. Decker documents test section construction and test pit observations in an internal report. Kohn focuses on the backcalculation results, which are covered in Chapter 4 of this report.

- The test pits dug in the MHB section (which utilized the guillotine hammer first) verified that the slab had broken full depth, with 9 to 12-inch maximum size pieces at the bottom of the PCC. However, test pits dug in the RPB section revealed some of the PCC below the steel had not broken, or had 24-inch plus maximum size pieces at the bottom. The pits also found substantial free water in the base layer.
- The RPB contractor (RMI) stated that the wrong RPB eccentric package was used here. Another factor that likely contributed to the RPB not being successful was the ASR that was prevalent in the PCC.

2.5   Specification Comparison

Existing agency rubblization specifications are compared in this section to evaluate different requirements contained therein. The organization and format of agency specifications vary, but most include similar headings for the different facets of rubblization. Comparisons are made here under the following headings: General Description, Materials, Equipment, Construction Requirements and Method of Measurement/Basis of Payment. The specifications reviewed for this comparison are listed in Table 2.3. These include the FAA's EB-66 P-215 [FAA, 2004], the USAF's ETL 01-09 [US AFCESA, 2001], and those of state DOTs that commonly rubblize PCC highway pavements. While other specifications exist, these provide an adequate representation. The FAA specification Item P-215 is the basis for comparison in this section.

While this section covers the content of various rubblization specifications, other sections of this report will build on this information to make recommendations for changes.

Agency-Year	Section/Item-Title URL
FAA-2004	Part of EB 66, P-215-Rubblizing Concrete Pavements http://www.faa.gov/airports_airtraffic/airports/construction/engineering_briefs/media/EB_66.pdf
USAF-2001	Part of ETL 01-09-Rubblizing Concrete Pavements https://www.afcesa.af.mil/userdocuments/publications/ETL/ETL%2001-9.pdf
Resonant Machines, Inc (RMI)	Separate specification recommendations for airport and highway rubblization http://www.resonantmachines.com/index.htm#
Arkansas Highway and Transportation Dept. -2003	Section 513-Rubblizing Portland Cement Concrete Pavement http://www.arkansashighways.com/Contract/progcon/general/STDSPECS_2003.HTM
Illinois DOT-2001	Construction Memorandum 01-40 http://www.dot.state.il.us/const/pdf/memorandum_40.pdf
Indiana DOT-2005	Section 305-Concrete Bases http://www.ai.org/dot/div/contracts/standards/book/sep06/3-2006.pdf
Iowa DOT-2002	Section 2217. Rubblizing Existing Portland Cement Concrete Pavement. http://165.206.203.37/APR_2006/GS/frames.htm
Louisiana DOTD-2000	Section 734 Rubblizing Portland Cement Concrete Pavement http://www.dotd.state.la.us/highways/project_devel/contractspecs/Part_VII.pdf
Michigan DOT-2003	Section 304 Rubblizing Portland Cement Concrete Pavement http://mdotwas1.mdot.state.mi.us/public/specbook/
Nevada DOT-2001	Section 410 Concrete Pavement Resurfacing http://www.nevadadot.com/business/contractor/standards/documents/2001StandardSpecifications.pdf
Ohio DOT-2002	Item 320 Rubblize and Roll http://www.dot.state.oh.us/construction/OCA/Specs/2002CMS/Specbook2002/300%20Bases/320.htm
Oklahoma DOT-1999	Section 433 Rubblizing Portland Cement Concrete Pavement http://fhwapap04.fhwa.dot.gov/nhswp/servlet/LookUpDivision
Pennsylvania DOT-2003 (mod)	Section 526 Rubblizing of Concrete Pavements ftp://ftp.dot.state.pa.us/public/bureaus/design/pub408/Change%20No6/Sections/526.pdf
Wisconsin DOT-2005	Section 335 Rubblized Pavement http://fhwapap04.fhwa.dot.gov/nhswp/index.jsp

Table 2.3, Rubblization Specification References

2.5.1 – General Description

The FAA (P-215) and USAF (ETL 01-09) specifications provide very similar descriptions, with the only difference of note being that P-215 refers to the possibility of "a new bituminous concrete or PCC pavement overlay," while ETL 01-9 only mentions a "new HMA overlay." Most state DOTs provide a similar description, with Pennsylvania distinguishing between "Type 1" and "Type 2" rubblization based on the maximum particle size allowed. Illinois also identifies different methods of rubblization in their specification, as will be described later.

2.5.2 – Materials

This part of the specification describes or references the requirements for materials used for patching or leveling rubblized PCC. P-215 includes a general description of patching materials and references Items P-401 (plant mix bituminous concrete) and P-209 (crushed aggregate base) for requirements. The volume of patching is determined in accordance FAA specification P-152 (unclassified subgrade excavation). State DOT rubblization specifications typically reference other specification sections for the requirements of crushed aggregate material used to replace old patches, repair test pits, reconstruct soft areas, or to raise the profile before placing the HMA overlays. HMA was also included for these applications in some state DOT specifications. ETL 01-9 includes requirements for patching or leveling materials under "Construction Requirements" and references other standards as well.

2.5.3 – Equipment

P-215 and ETL 01-9 have identical language describing both types of breaking equipment (RPB and MHB) and seating (rolling) equipment. Both are also very similar to most state DOT specifications, although Iowa, Pennsylvania and Wisconsin do not include detailed descriptions of equipment. Nevada specifically disallows the use of a "guillotine type impact hammer, equipment that punches holes in the pavement, or unguided free-falling weights such as 'headache' or 'wrecking' balls." Iowa requires that the process "be capable of uniformly breaking the existing pavement without causing significant change to its cross slope or profile."

Most of the state DOT specifications describe and allow both types of equipment, although Arkansas, Louisiana and Utah only describe and allow the RPB. Illinois describes four different methods (I, II, III and IV) using one or both equipment types that must be specified on the plans, depending on slab thickness and support conditions. Method I uses the MHB and Z-grid roller, Method II uses the RPB with high flotation tires (tire pressure less than 60 psi), Method III uses the RPB without restricting the tire pressure, and Method IV allows either device. The Illinois DOT approach is discussed further in Chapter 3 of this report.

Chapter 6 also discusses rubblization and seating equipment.

Resonant Pavement Breaker

The language used to describe the resonant pavement breaker (RPB) in both airfield rubblization specifications is as follows: *This is a self-contained, self-propelled resonant frequency breaker specifically designed for the purpose of rubblizing PCC pavement. The machine shall be capable of producing low-amplitude (25 millimeters [1 inch] maximum) blows*

of 8.9 kilonewtons (2000 pounds force), and delivering blows to the existing PCC surface at a rate of not less than 44 cycles per second. If necessary, the breaker shall be equipped with a screen to protect nearby structures, vehicles or aircraft from flying chips during the fracturing process. This description generally follows RMI's recommendations and is essentially the same as most state specifications. One difference is that RMI allows for a range of frequencies (42-46 Hz) and for up to a 1.25-inch amplitude to include the differences in the new, larger model PB-500 compared to the PB-4 that has been used extensively on highways.

Multi-head Breaker

For the multi-head breaker (MHB), the following description is provided in both airfield rubblization specifications: *This is a self-contained, self-propelled multi-head breaker specifically designed for the purpose of rubblizing PCC pavement. The machine shall be capable of rubblizing the pavement a minimum width of 3.9 meters (13 feet) per pass. Pavement-breaking hammers shall be mounted laterally in pairs, with half the hammers in a forward row and the remainder diagonally offset in a rear row so there is continuous breakage from side to side. The lift height of the hammers shall be independently adjustable. If necessary, the breaker shall be equipped with a screen to protect vehicles from flying chips during the fracturing process.* The Ohio DOT specification provides a more detailed description of the MHB, including the ranges in weight and applied force for the drop hammers, the frequency of impacts (30-35 blows per minute), and the maximum lift height (60 inches) of the weights.

Rollers

Most of the specifications reviewed include roller equipment requirements used for seating and/or fracturing the rubblized material. The actual prescribed rolling sequence is covered later in Section 2.5.4. A smooth drum vibratory roller with a gross weight of not less than 10 tons is recommended for seating and described in all but one of the specifications, regardless of the type of rubblizing equipment employed. Pennsylvania is the exception and does not allow vibratory rollers on rubblized concrete. Both the FAA and the RMI specifications call for "dual" or "tandem" when describing the vibratory steel drum roller for use with RPB. Other specifications do not. We do not believe using the vibratory steel drum roller needs to include two drums when using the RPB or the MHB. Although both specifications refer to the Z-Grid Roller as "seating" equipment for use with the MHB, it is actually part of the fracturing process and would be more accurately described as such.

Both airfield rubblization specifications require the use of a pneumatic-tire (rubber-tire) roller with a gross weight of at least 25 tons for seating the fractured concrete when using the MHB. Some highway specifications mandate the use of a heavy (35 to 50 ton) rubber-tire roller when rubblizing with the MHB, but there are no consistent requirements, such as seen for the steel drum vibratory rollers. The FHWA specification allows either a 35 or 50 ton roller but requires a different rolling pattern depending on the mass of the roller. The Illinois requirement is based on the static linear load of the rubber-tire roller, rather than on the total mass of the roller. The Nevada specification provides the most detailed description of rubber-tire rollers, setting requirements for minimum roller mass (30 tons), numbers of wheels (7) and axles (2), width between the outer edges of outside tires (5 ft), maximum variation in tire pressure (5 psi), and minimum tire pressure (150 psi). Unlike other specifications, P-215 allows the use of equipment of "other design that will accomplish similar results," as approved by the engineer.

2.5.4- Construction Requirements

This section of P-215 describes work to be performed as preparation for rubblization, test strip/test pit requirements, rubblization criteria and processes, seating procedures and miscellaneous requirements (patching, cutting off exposed steel, etc.) Preparatory work includes installation of drainage systems, removal of existing asphalt surfaces and asphalt patches, full-depth saw cutting to isolate the concrete to be rubblized from adjacent concrete to remain intact, and any shoulder requirements.

Drainage System
Installation or replacement of longitudinal edge drains is common on highway rubblization projects. P-215 and ETL 1-09 both state that any drainage system installed should be "properly functioning for a minimum of two weeks prior to rubblization." RMI in their highway specification suggests that an edge drain system be installed at least a week before rubblizing. Of the state DOT specifications reviewed, about half required the installation of a functioning drainage system before rubblization (IL, IN, LA, MI), others did not include such a requirement but did direct the contractor to avoid damaging any drainage facilities already in place (OK, PA), while others did not mention the edge drain system within their rubblization specification.   Details and construction requirements for edge drains are included in other specification items.
Chapter 6 discusses drainage systems in more detail.

Removal of Existing Asphalt Surfaces/Patches
There is general agreement that any asphalt overlay must be fully removed to allow the breaking device to fracture the concrete pavement. Some agency rubblization specifications (FAA, USAF, AR, IL, OK & PA) state that an existing overlay must be removed before rubblizing while other state specifications do not mention such a requirement because any such requirement is called for in the project plans.
While P-215 states that existing full-depth asphalt patches are to remain in place, many of the rubblization specifications reviewed did not mention them. Of those that did, there was no general agreement on whether they should be removed. Arkansas and Illinois indicate that the contractor will be told which patches are to remain and which are to be removed. Oklahoma requires any asphalt patch greater than one square meter to be removed. Pennsylvania states that any existing full-depth bituminous patch should remain. .

Saw-Cut Joints
Most rubblizing specifications use similar language to describe the separation of the rubblized concrete pavement from adjacent concrete pavement that is to remain intact. Specifications require a full-depth saw-cut along an existing joint and require any load transfer devices or tie bars be severed to prevent damage to adjacent non-rubblized sections.

Shouldering
Where specifications mention shoulder work in preparation for rubblizing, they all use language similar to P-215, which requires the shoulder or widening work be brought to the same level as the rubblized pavement surface.

Test Strip/Test Pits

P-215 specifically describes the test strip as 150 ft by 12 ft whose location is designated by the Engineer and where the contractor will adjust the equipment and/or operation to yield the desired result as determined from excavating a test pit. Most of the state DOT specifications reviewed did not mention a test strip. Ohio and Oklahoma both note that a test strip will be designated by the Engineer but did not provide specific dimensions. Nevada requires a test strip at least a full lane wide and 100 ft long. RMI's airport specification suggests the test strip be at least 300 ft by 8 ft.

Test pits are excavated to determine whether the fracturing process is producing particles of the prescribed dimensions and is debonding the concrete from any steel. P-215 describes test pits similar to the Arkansas, Louisiana and RMI specifications: 4 ft by 4 ft in the middle of the test strip as designated by the Engineer. Nevada and Pennsylvania require the test pit be the width of a typical highway lane, while others did not describe any dimensions for test pits. The FHWA specification does not require a test pit, but rather a test section from which ten 6-inch diameter cores are taken to verify the slab is fractured full-depth.

The P-215 specification calls for the Engineer and contractor to mutually agree on the process and equipment to be used throughout the project based on test pit results. It also notes that additional test pits might be required to confirm that the concrete is being completely fractured. Some agencies only call for a test pit on the first day of work or in the test strip (AR, OH) while others require test pits each day (NV) or at distance/production intervals (MI, WI). Michigan requires test pits every 1500 ft.

A likely reason for these differences is the way different agencies describe general acceptance requirements within their standard specifications. For example, FAA AC 150/5370-10B (dated 4/25/2005) includes Sections that define the roles and responsibilities of Engineers, Inspectors and Contractors. Section 50 (Control of Work) and Section 60 (Control of Materials) could be cited when requesting the excavation of a test pit and evaluation of materials therein beyond the schedule defined within a rubblizing specification.

Chapter 5 of this report cover test strips and test pits in more detail.

Rubblization Criteria

This section provides the basis for accepting a rubblized PCCP. Several specifications, including P-215, have different particle size requirements depending on depth/location within the slab (surface/upper half/above reinforcement, bottom half/below reinforcement, near edges). This type of criterion appears to be more common where the MHB is anticipated because states that exclusively use the RPB appear to have borrowed terminology directly from RMI's highway guide specification (*"broken into pieces ranging from sand size to pieces generally 6 inches or less in size,"* and *"majority of particles between 1 and 3 in"*). USAF ETL 01-09 states that particles will generally be larger with the MHB versus the RPB and has a different maximum particle size criterion, allowing a maximum size of 9-inches with the MHB versus 8-inches with the RPB. The airfield specifications also allowed for larger particles near the edge of the pavement versus interior.

P-215, ETL 01-09 and the RMI airfield specification allows larger particles than most highway specifications. RMI suggests that *"The existing pcc pavement shall be broken into pieces from sand size to pieces not greater than 1.25 times the maximum depth of the slab. The majority of pieces shall be from sand size to 0.75 times the maximum depth of the slab."*

Referencing the existing slab thickness when setting particle size requirements provides flexibility in the specification since airfield PCC pavements vary widely in thickness, and often include much thicker slabs than typical highway pavements.

Rubblization specifications generally require that the concrete be debonded from any reinforcing steel. Michigan requires the use of "manual methods" to excavate the PCC material down to the reinforcement until adequate debonding has been determined.

Chapter 5 covers particle size requirements in more detail.

Rubblization Procedures

While the specifications reviewed were not always consistent in their organization of addressing various issues related to rubblization procedures, most had similar verbiage on these issues as P-215. They include:

- Debris must not enter active pavement.
- Water trucks may be used to control dust, but excessive water should not be applied to the rubblized surface.
- The rubblization procedures should avoid damaging the base, underlying structures, utilities and drainage facilities. If damage does occur, the contractor must notify the Engineer and is responsible for the repair.
- Reinforcing steel should generally be left in place, except that any steel becoming exposed during the rubblization and rolling process should be cut flush with the surface.
- Joint seal or joint filler material can also remain in place.

In the case where an HMA overlay is placed directly over rubblized PCC and is adjacent to a pavement that has yet to be rubblized, P-215 requires that the rubblization extend at least 2 feet beyond the edge of the new HMA overlay. Most other specifications require only a 6-inch extension.; creating a greater likelihood the subsequent rubblization on the adjacent area may damage the new HMA.

Seating/Rolling Procedures

This refers to the methods used to roll the fractured concrete, which differ for each type of rubblizing equipment. For concrete broken with a RPB, P-215 requires a minimum of three passes (front and back defined as a single pass) over the entire width of broken concrete using a vibratory steel drum roller. A maximum roller speed of 6 feet per second is stipulated. For the MHB process, P-215 requires at least two passes using a Z-grid roller, followed by a minimum of one pass using the pneumatic roller. At least one pass using a vibratory steel drum roller is also mandated immediately before placing the first lift of HMA. P-215 allows the Engineer to require additional roller passes for either type of rubblizing equipment.

Most of the other rubblization specifications call for a specified rolling pattern using both vibratory steel drum rollers and pneumatic rollers that meet specified equipment requirements. The details varied somewhat, but most specifications required a total of three passes using a vibratory roller, with the last pass immediately before paving and at a speed less than 6 feet per second. No specification reviewed placed a limit on the amplitude of the drum, and only one (IN) required a minimum frequency (2000 vpm). Michigan allows the application of water to the surface of the rubblized concrete to aid in reorienting particles during rolling, but only before the third roller pass. P-215 cautions against rolling MHB-broken concrete in wet conditions. Pennsylvania differed from others by requiring a seating (50- ton rubber tire) roller and not allowing the use of vibratory rollers.

Chapter 6 builds on this information to discuss rolling recommendations.

Unstable Area Patching

P-215 includes full depth patching requirements for two different conditions. The first is for unstable areas caused by "expansion of the existing concrete pavement." These call for a full-depth HMA patch placed before the overlay no larger than 4 feet long and 12 feet wide. No other specification reviewed discussed patching requirements for these "expansion" conditions. The second condition is areas of poor subgrade support found during the rubblization or seating process. P-215 handles these unstable areas similar to others specifications, removing the rubblized pavement, base course and subgrade at the direction of the Engineer and replacing with either aggregate base or HMA. Current P-215 verbiage appears to be unclear regarding payment for these full depth patched areas. Some state specifications offer more definitive guidance on full-depth patching, and project plans include an estimated quantity of full-depth patches.

Minor surface depressions after rubblizing and rolling are addressed in some highway specifications, but not in P-215. Generally, shallow depressions that are considered stable are filled with crushed aggregate and shaped before rolling operations.

Chapter 3 provides an extensive discussion on dealing with unstable areas and Chapter 6 further discusses full depth patching of unstable areas.

Relief Trenches

While EB-66 discusses the possible need for cutting relief trenches when utilizing the RPB to allow for horizontal expansion of the rubblized PCC, P-215 does not mention this further. RMI's airfield specification also includes discussion of relief trenches. Thicker slabs and smaller resonant machines are more likely to need relief trenches. Highway specifications do not address this issue because roadway pavements are much narrower and usually thinner than those sites where relief trenching has been necessary.

Relief trenches are also discussed in Chapter 6.

Surface Smoothness or Grade Control

No provisions were found in any of the rubblization specifications reviewed for surface smoothness or grade control requirements. This is likely because a rubblized surface cannot be finish graded like soil or aggregate bases. Aggregate bases may be placed on top of rubblized surfaces (section 3.3.6).

2.5.5 - Method of Measurement/Basis of Payment

Rubblization is consistently measured and paid for by unit area (yd^2).

CHAPTER 3 – ASSESSING PAVEMENT SUITABILITY

With approximately 50 million square yards of PCCP having been rubblized on hundreds of projects in the U.S. this past decade, it is clear that this rehabilitation technique has been tremendously successful (see Chapter 2). Due to it's effectiveness in eliminating slab action and advantages of cost, speed and utilization of in-place materials, rubblization has become the most popular and effective method of converting a failed PCC pavement (PCCP) into a new flexible pavement system. Airfield pavements ranging in thickness from 6 to 26 inches have been successfully rubblized (Tables 2.1 and 2.2). That said, not all PCC pavements are good candidates for rubblization.

This chapter addresses how to assess the suitability of a PCCP for rubblization and provides recommendations when dealing with marginally stable conditions. The feasibility of a project for rubblization depends on certain physical factors of that PCCP system that relate to its ability to be suitably broken or fractured to desired criteria. Of particular interest here are airfields where the slabs are relatively thin (less than 9 inches), have weak underlying support with little or no subbase, and soft or saturated subgrade. Some World War II era airfields built in the U.S. and later converted into general aviation (GA) airfields fit into this category. The combination of thin slabs with weak support (often caused by poor drainage) produces unstable areas that may lead to difficulty in effectively fracturing the PCC to specified criteria. These difficulties were discussed in Section 2.4 for three runway projects: Pratt Airport, Kegelman Field and Tullahoma Airport, and to a lesser degree at the Grant County airport.

Section 3.1 discusses some general physical considerations of the PCCP other than slab thickness, underlying support and drainage that can inhibit the rubblization process. Section 3.2 addresses the issue of how to assess the feasibility of rubblizing these thin, marginally stable pavements. Criteria, evaluation methodology and decision protocol currently used by other agencies is presented, along our research team's recommendations. Section 3.3 provides other recommendations that should be considered during the planning, design and construction phases for rubblizing marginally stable pavements in order to maximize success.

3.1   General PCCP Physical Considerations

Slab thickness and subbase support are the dominant factors which affect the ability to successfully rubblize a PCC pavement with today's equipment. Often, poor drainage causes the weak support conditions that inhibit the rubblization process. While Section 3.2 probes these factors in detail, this section first looks at how other physical factors of the PCCP can inhibit the fracturing process. These factors are PCCP type, distress conditions and aggregate hardness.

3.1.1   Type

Over the years, there have been a multitude of PCCP types built that are differentiated by the type and existence of transverse joints, load transfer devices, and temperature steel. Plain-jointed PCCP with and without dowels and plain-jointed PCCP with welded wire mesh for crack control are probably the most common PCCP types used on airfields. All these PCCP types, including jointed-reinforced concrete pavements (JRCP) and continuous-reinforced concrete pavements (CRCP), have been successfully rubblized over a wide range of thicknesses and

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environmental conditions. However, the presence of steel will typically hinder the fracturing process by absorbing or dissipating part of the energy induced by the rubblization equipment. The amount of fracture is generally less around and below any steel layer and/or dowel bars, as was seen in the Michigan study (Section 2.2.3, Figures 2.1 and 2.2, and Section 4.2.6), the Detroit Airport Trial (Sections 2.4 and 4.2.8) and the FAA's rubblization trial at NAPTF (Sections 2.2.3 and 4.2.9, Figure 4.4 and Appendix A). Rubblization specifications typically account for this fact with less stringent particle size criteria below any reinforcement steel (Section 2.5.4). The quality assurance recommendations provided in Sections 5.3 and 5.4 of this report on how the test pit criteria in a specification should handle reinforcing steel and dowel bars also considers this fact.

Thickened edge PCCP, often found on highway pavements such as the 9-6-9 type, may offer a challenge to providing a consistently fractured slab due to the variation in slab thickness. This observation was made on one recent highway trial project in Texas with 9-6-9 pavement where the MHB was used. The RPB may need to adjust it's fracture energy with each pass (generally less than a foot wide) while the MHB, with it's adjustable 2.67 to 13-feet breaking width, may need to adjust the drop height on the hammers impacting directly over the thickened slab edge (at the joints). While this is a concern, it should be stated that states such as Iowa and Wisconsin rubblize thickened edge highway pavements routinely without documented problems with both pieces of equipment. Thickened edge PCCP is also found on airfields where separate pavement features meet one another and additional load transfer is necessary.

3.1.2　Distress Conditions

In severe cases of certain material distress types, the condition of the PCCP may affect the consistency of fracture with the rubblization process. Severe conditions of material-related distresses such as D-cracking and alkali-silica reactivity (ASR) may inhibit consistent fracture through the entire slab. This is because some of the fracture energy could be absorbed at the top of the "soft" concrete that has severe internal cracking. Even though this phenomenon could occur, pavements with significant ASR and D-cracking are often cited in DOT guidance as prime candidates for rubblization. Both the MHB and RPB have successfully rubblized PCCP suffering from severe ASR. Two projects that illustrate this, and have already been discussed in this report, are Colorado I-76 (see Section 2.2.3) and the Detroit Metro Airport Demo (see Section 2.4).

An additional performance concern recently raised within the U.S. military airfield pavement engineering community is that rubblizing PCCP with ASR may allow an opportunity for secondary chemical reactions of the rubblized concrete, which in theory could cause enough vertical swelling within the rubblized layer to create a smoothness problem on the pavement surface. As part of this project, this research team explored this issue in detail and found zero evidence in the 25 years of performance history with fractured slab technology to warrant such a concern. This issue is fully addressed in Section 6.11.

3.1.3　Aggregate Hardness

According to conversations with contractors, it is more difficult to rubblize PCC that contains very hard coarse aggregates. Concrete produced with very hard aggregates such as crushed siliceous river gravel usually fracture around, rather than through, aggregate particles

when specimens are broken in a laboratory. The same breaking pattern appears to take place during rubblization in PCCP with very hard aggregates, increasing the length over which the crack must propagate to extend through the entire slab. This phenomena was theorized as a contributing factor to why the 19-inch (and thicker) slabs on the Grand Forks AFB Runway project were slow to rubblize with the RPB (Section 2.4).

3.2   Assessing Weak Pavements

With rubblization, slabs are fractured by being placed in tension. If the overall pavement system is not stiff enough, the fracturing process can be inhibited. Project experience has shown that relatively thin PCCP (less than about 9 inches) and with poor underlying support may fall in this category. The presence of free water exacerbates the problem. Unfortunately, many candidate rubblization projects have free water directly below the slabs, which was a major contributor to the pavement's deterioration in the first place. One primary purpose of installing edge drains, covered in Chapter 6, is to remove this free water prior to rubblization, which will help immensely in providing a more consistent fractured layer.

While there are three runway projects discussed in Section 2.4 (Pratt, Kegelman and Tullahoma) that experienced stability problems that significantly affected the project, there have also been many rubblization projects with slabs between 6 to 8-inches thick that were deemed very successfully. Tables 2.1 and 2.2 reveal 13 completed airfield rubblization projects with slab thicknesses between 6-8 inches. Two runway projects, Rantoul Airport and Grant County Airport, are discussed in Section 2.4 and had slabs as thin as 6-inches. On the Grant County project, only one rather small area (100 ft by 200 ft) had to be replaced with a full-depth patch. It is important to note that the 6-inch slabs at Grant were supported by a 7 to 15 inch thick subbase described as marginal quality on a subgrade design CBR of 7. This is much better slab support than at Pratt, Kegelman or Tullahoma.

In this section, the only criterion found in the literature for defining unstable areas (from Wisconsin DOT and Illinois DOT), is presented. Evaluation methodology and decision criteria established for Texas DOT, which was derived from the Illinois criteria, is then explained and is followed by a case study. Finally, our research team's recommended approach is presented for predicting areas on airfield that will likely be too unstable to successfully rubblize.

3.2.1   Wisconsin Criteria

Wis DOT is of particular interest due to the variety of concrete pavement types in that state and their considerable experience with rubblization. Their published guidance [Wis DOT, 2003] states, "PCCP directly on weak subgrade makes rubblization susceptible to subgrade yielding problems." To help detect these conditions prior to construction, they suggest a subgrade investigation to determine the CBR using the Dynamic Cone Pentrometer (DCP). The chart shown below in Figure 3.1 is then used to determine "whether the subgrade, rubblized concrete and base course provide enough support to accommodate construction traffic, or if remedial action is required." The y-axis here is the total thickness of both the slab and any underlying base. If "remedial action" is required, they suggest placing an additional dense aggregate or RAP base course layer above the rubblized concrete. One should note that while this will help with providing a more stable working platform for trucking and placing the first lift

of HMA, it does not address the instability problems that can occur during the rubblization process itself (rutting and punch-thrus with the RPB, ineffective fracture with both the RPB and MHB). The guidance goes on to state that the specification should allow for "excavation below subgrade" (full depth patching) and "modification of the broken particle size requirements in localized areas" where there is poor stability.
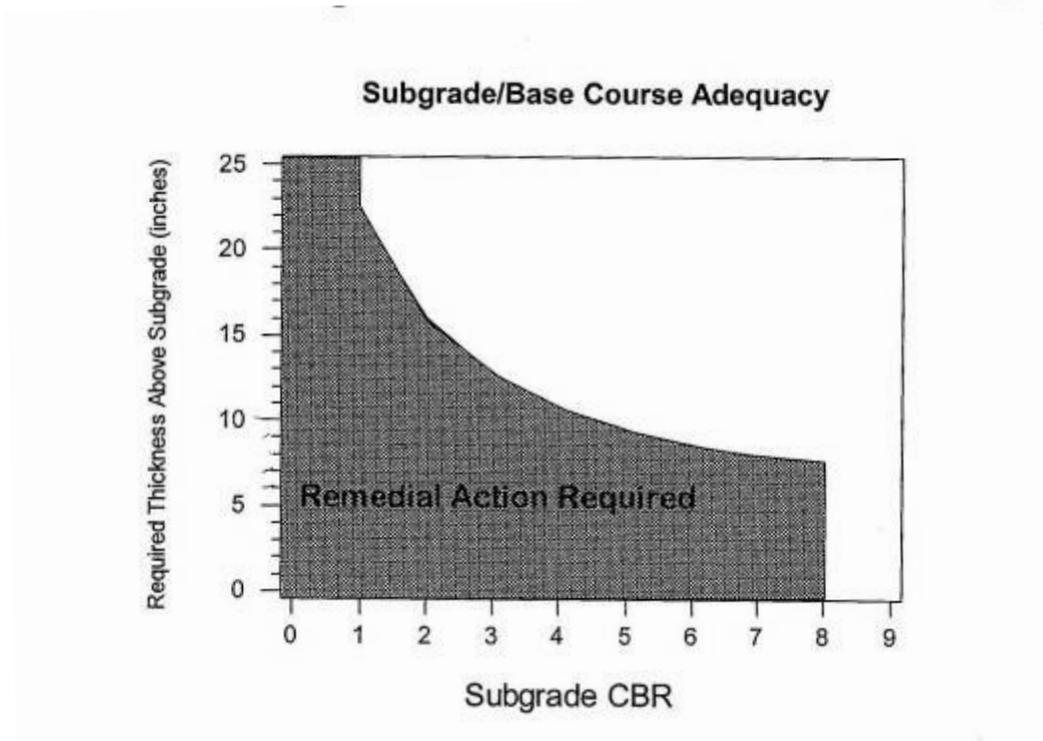
Figure 3.1, Wisconsin DOT Criteria for Predicting Unstable Areas [Wis DOT, 2003]

3.2.2   Illinois Criteria

The Illinois DOT developed a chart shown in Figure 3.2 for determining whether rubblizing was a feasible alternative, and which equipment methods could be used [Heckel, 2002]. The chart's y-axis is total pavement thickness (slab plus any base layer) while the x-axis is the subgrade Illinois Bearing Value (IBV). The IBV is very similar to the California Bearing Ration (CBR). The IBV is determined directly from DCP testing in accordance with Illinois Test Procedure 501 [ILDOT PTA-T4, 2005). A review of this standard reveals that the DCP equipment is identical to the one used by the U.S. Army Corps of Engineers (COE). Even though the ILDOT equation that calculates IBV from the penetration resistance (PR) of the DCP is different from the COE equation that calculates CBR from the DCP, the IBV and CBR are almost identical given the same PR. This is shown in Table 3.1.
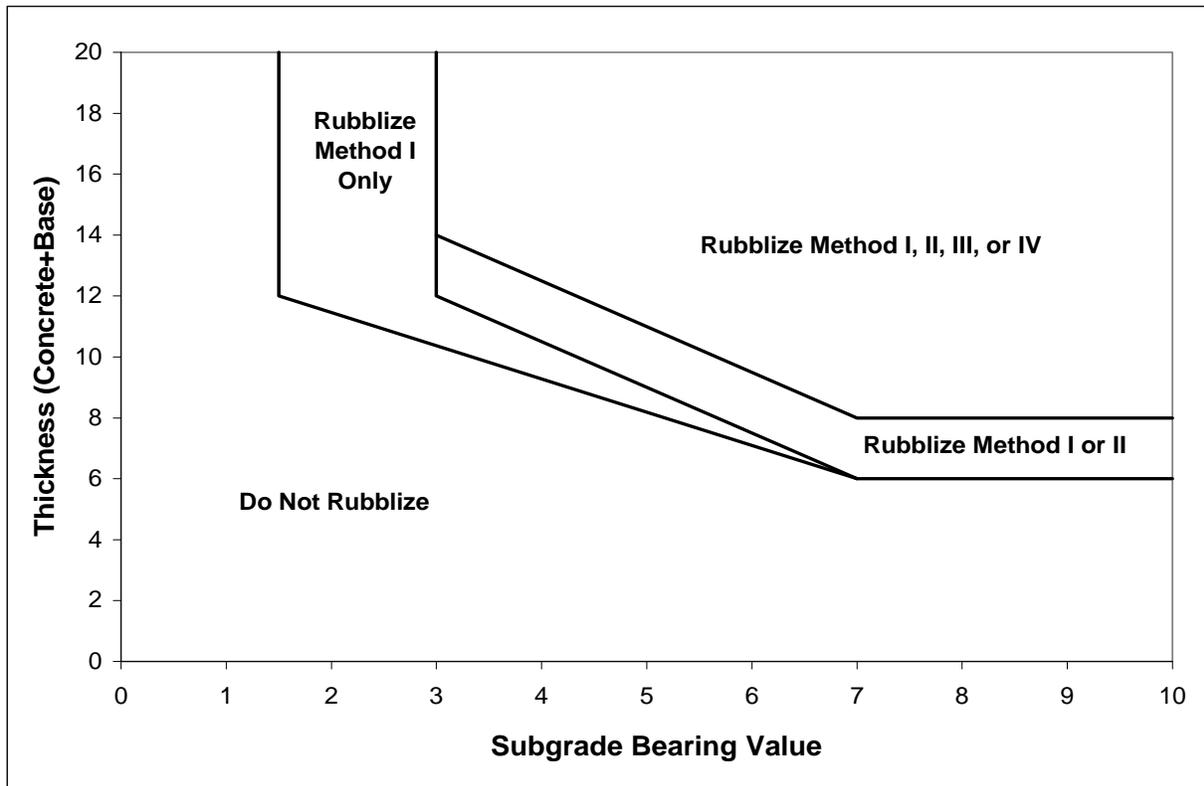
Figure 3.2, Illinois DOT Rubblization Selection Chart [Heckel, 2002].

PR from DCP (mm/blow)	Calculated CBR	Calculated SBV
5	48.1	53.4
10	22.2	22.3
15	14.1	13.4
20	10.2	9.3
25	7.9	7.0
30	6.5	5.6
35	5.4	4.6
40	4.7	3.9
45	4.1	3.4
50	3.7	2.9
55	3.3	2.6
60	3.0	2.3
65	2.7	2.1
70	2.5	1.9
75	2.3	1.8

Table 3.1, Comparison between Illinois DOT IBV and COE CBR from DCP Data

The rubblization methods designated by ILDOT include:
- Method I: MHB.
- Method II: RPB high flotation (HF) tires with tire pressures less than 60 psi.
- Method III: RPB with no restriction on tire pressures.
- Method IV: MHB or RPB.

This approach was proposed by Thompson based on stress ratio computations using ILLIPAV with the different loading configurations of the rubblization equipment [Thompson letter, 2001]. The ILDOT chart in Figure 3.2 recommends that only the MHB be used (Method I) when the slab and base thickness is greater than 12 inches and the subgrade bearing value (or comparable CBR) is between 1.5 and to 3. The rationale is that the multiple passes required by the RPB may cause structural damage to the section, whereas the single pass of the MHB will not cause damage because it does not operate on broken concrete.

3.2.3   Texas DOT Evaluation Methodology and Decision Criteria

In 2004, the Texas DOT started a research program to develop selection criteria for potential rubblization projects. The initial rubblization projects in Texas had been considered unsatisfactory because in one case, 20% full-depth replacement was required and in another, the project was abandoned because of trapped water in the broken concrete.

Researchers at the Texas Transportation Institute [Sebesta et al, 2006] surveyed the available literature, found the ILDOT procedure, and built it into a proposed TxDOT evaluation plan. This plan collects information on the pavement structure through historical records, a visual condition survey, and in-situ testing that includes ground penetrating radar (GPR), falling weight deflectometer (FWD), and dynamic cone penetrometer (DCP). A GPR survey is used to estimate pavement layer thicknesses, identify changes in the pavement structure, and detect locations of wet subgrade or saturated voids. The FWD provides data to evaluate the structural condition of the pavement layers and overall pavement strength at one location relative to others. The DCP data serves for validation of base layer thicknesses and base and subgrade strength conditions.

Highlights of the data collection used as part of the Texas evaluation plan are:
- Plans: Collect and review plans from the project to identify the existing pavement structure. Identify important parameters such as existence of any treated subgrade layers, type and thickness of any base, slab thickness, type of any steel, and thickness of any HMA overlays. Review the available soils information (USDA maps), check soil type and reported water table locations.
- Visual Condition Survey: Review project for the types and severity of distresses present. Note the location of any maintenance actions where the structure may be different. Look for signs of areas with poor drainage and poor subgrade conditions such as base pumping.
- GPR: Survey the entire project. GPR is used primarily to locate areas of trapped moisture either around joints or in the subgrade. GPR can also detect non-uniformity of the pavement structure, especially if it has several asphalt overlays. It can show where slabs have been replaced. GPR data is used to estimate existing pavement layer thicknesses, locate limits of potential section breaks in the pavement structure and identify air filled and water filled voids. In Texas a 1 GHz air coupled GPR system is

used to test candidate pavements. However, if this equipment is not available, it should be possible to collect similar information with a ground coupled GPR system operating at a similar frequency.

- FWD: Collect FWD data on the project at a maximum spacing of 0.1 miles for each traffic lane, or at intervals sufficient to obtain at least 30 drops on the project. Closer testing intervals are permissible. Collect the drops in the center of the concrete slabs. If the project is jointed concrete and an asphalt overlay option is being considered as an alternative to rubblization, then also perform load transfer tests across the joints.
- DCP: From the FWD data, identify the locations with the highest and lowest deflections at the outermost deflection sensor. At these locations, core through the concrete to perform DCP tests starting at the top of the base. If the base is stabilized, drill through the base and start at the weaker support layers. Conduct DCP tests in a minimum of three locations with high outer FWD sensor deflection (weak area) and at least one location with low outer sensor deflection. Select two additional locations for a minimum of six DCPs. Determine the thickness of any base and estimate the base and subgrade CBR using the COE procedure. If the DCP data does not clearly detect a base, then use the CBR of the first 6 inches below the slab as a "dummy" base layer, and determine the subgrade CBR of the next 6 inches.

Once the evaluation data is collected, the project is analyzed with the following steps:
- Using the DCP data, plot the slab thickness versus the base CBR on the ILDOT chart in Figure 3.2. These data give an indication of whether the concrete will rubblize since sufficient support beneath the slab is crucial for satisfactory breakage.
- Plot the combined thickness of the slab-plus-base versus the CBR of the subgrade. Use a "dummy" base layer of 6-inches if the DCP does not reveal a base layer. These data give an indication whether the subgrade can support construction traffic after rubblization.
- If all the data points fall in the zones indicating rubblization is feasible, the pavement is suitable for rubblization. If all the data points fall in the "Do Not Rubblize" zone, other rehabilitation options should be considered.
- If some but not all the data points fall in the "Do Not Rubblize" zone, certain portions of the project may not be suitable for rubblization. More analysis, interpretation and judgment are required. Perform the following analysis.
    - Determine the average CBR of the first 12 inches beneath the slab.
    - From the ILDOT chart, determine the minimum CBR necessary to support rubblization for the known slab thickness. Do this by starting at the Y-axis at the known slab thickness and project horizontally until intersecting the boundary where rubblization is feasible. At this intersection, project down to the X-axis and read the minimum CBR required.
    - In order to minimize coring and DCP testing, form a relationship between the subgrade modulus obtained from FWD testing and the CBR results obtained from DCP testing at the stations where both were conducted by graphing the average CBR of the first 12 inches below the slab versus the subgrade modulu. Input the minimum CBR required for rubblization to determine the comparable minimum subgrade modulus for rubblization.
    - Graph the subgrade modulus versus station number for the project. Where the modulus does not exceed the minimum required, there is risk that the pavement

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may not effectively rubblize. Even on projects where the pavement section appears to be uniform, some areas will rubblize while others may not due to subtle differences in subgrade and drainage conditions. Engineering judgment is necessary at this point, but it is helpful to have an estimate of unstable areas and also to know where the weakest support conditions may be expected.

3.2.4   Case Study Using Texas Methodology

The TX DOT evaluation method has recently been applied to a number of candidate sections in Texas. The candidate pavements built since 1960 have treated bases and have no problems passing the ILDOT criteria. Older pavements, however, particularly the thickened edge 9-6-9 type, have weak areas predicted to be problematic. One such pavement section, on Texas FM 1155 from FM 912 to Park Road 12, is illustrated here as a case study to demonstrate the TX DOT methodology. This pavement section is a rural two-lane highway with joint spacing every 40 feet and transverse cracking about every 7 to 10 ft.

Figure 3.3 shows a typical display from the GPR survey showing the cross-section for about 400 ft of this project. The pavement surface is the horizontal line at the top of the figure, the depth scale is in inches on the right, and the distance scale is in miles and feet at the bottom. To the left of station mark 0 miles-5238 feet is a long section that was patched with full-depth asphalt. To the right of station mark 0-5238, the bottom of the concrete is evident just below a depth of 6 inches. Increases in reflection energy show up as stronger reflections on the plots, which are indicators of layer interfaces as well as wet areas under the slab. To judge the suitability for rubblization, it is important to note the intensity of the reflection at the bottom of the slab to estimate the presence of wet and voided areas.
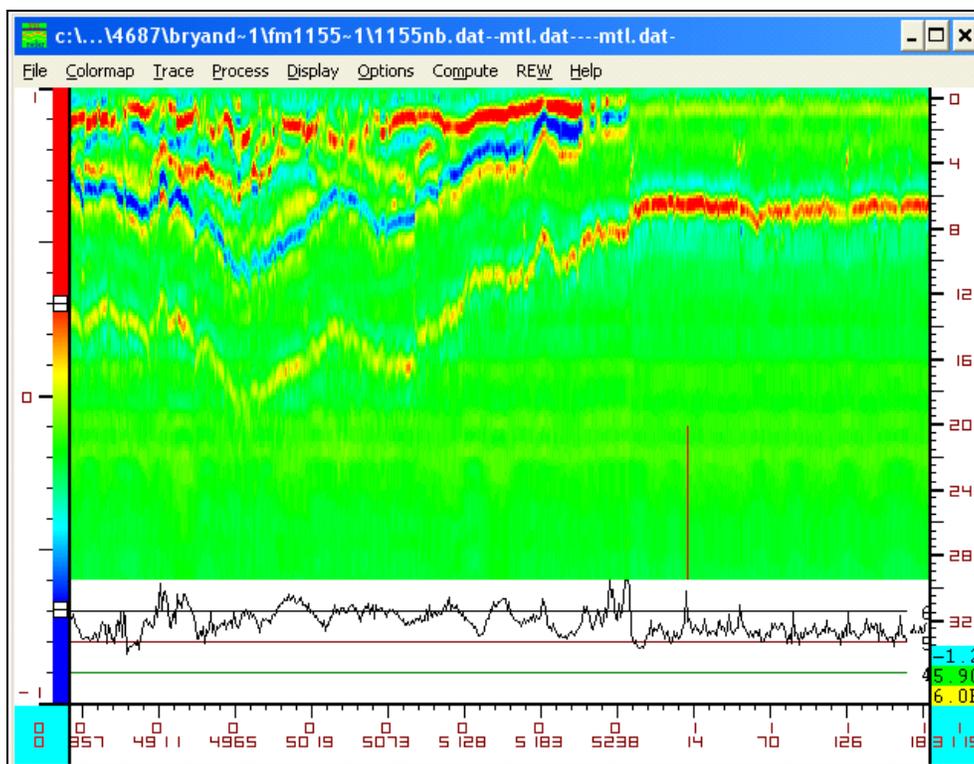
Figure 3.3, Example GPR Data from Texas FM 1155

An FWD survey was completed with drops collected on the center slab every tenth of a mile. From the FWD results, seven locations were selected for DCP testing. Since construction records and the DCP results indicated there was approximately six inches of select fill material as a base layer, a CBR was computed for this layer as well for the depth of 6 to 12 inches below the slab. The results from the DCP testing are shown in Table 3.2.

DCP Test Location	Concrete Thickness (in)	Base Thickness (in)	CBR Values		Subgrade Modulus from FWD (ksi)	Location (feet North from RM 631 on FM 912)
			Base	Subgrade		
1	7.5	6*	5.5	3.8	8.2	500
2	8.5	6*	66	133.0	12.7	1002
3	7.4	4.3	15.5	7.5	10.1	3006
4	7.0	6*	15.8	12.6	5.3	3260
5	7.0	6*	3	2.6	7.1	5000
6	7.1	5.9	9.0	2.2	8.9	6750
7	7.4	4.1	8.5	2.4	7.7	7257

*Assigned to 6 inches because no clear base layer boundary observed in DCP data

Table 3.2, DCP Results from Texas FM 1155

The results in Table 3.2 indicate the support strength below the slab is quite variable and very poor in some areas. The DCP results were then plotted on the ILDOT chart as shown in Figure 3.4. Each DCP location has a CBR value directly below the slab (designated as base) and another at the top of the subgrade (just below the base or designated base). The diamonds in Figure 3.4 are the slab thicknesses plotted versus base CBR while the triangles are the slab-plus-base thicknesses plotted versus subgrade CBR.
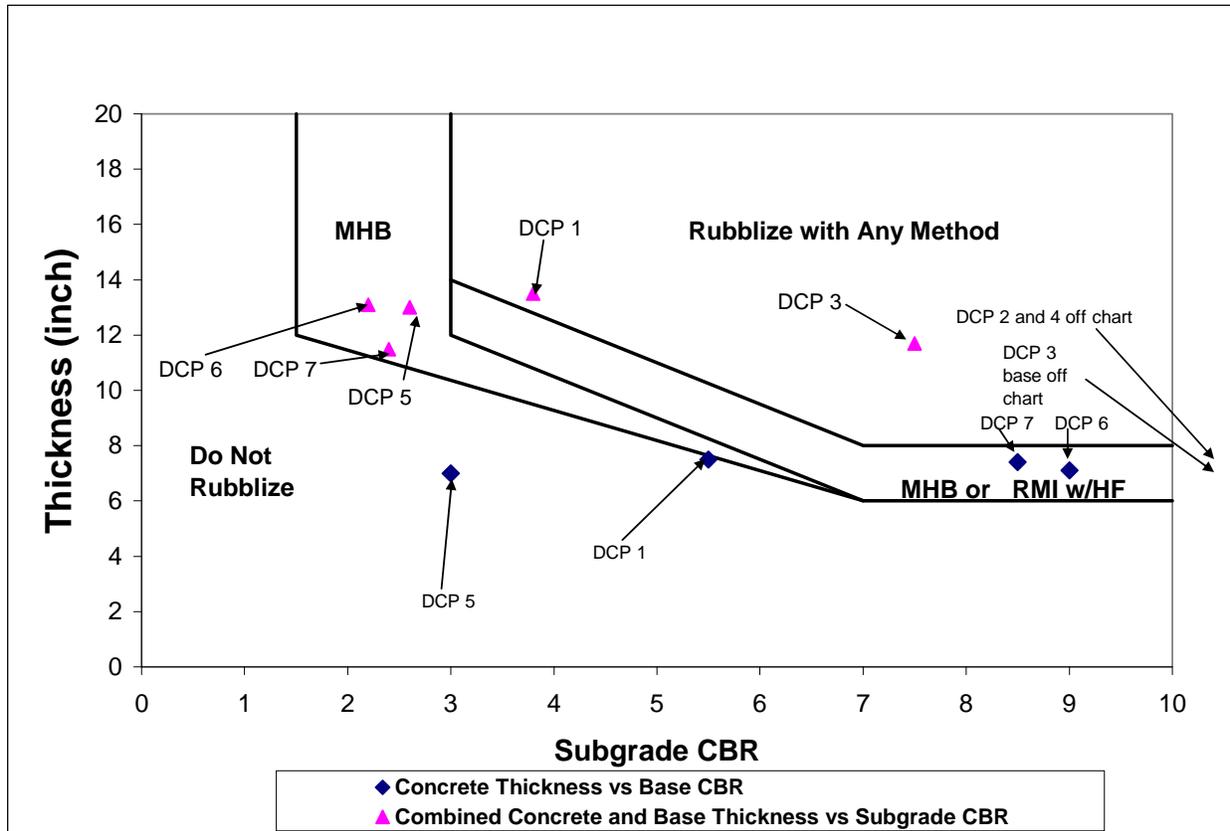
Figure 3.4, Plotting Texas FM 1155 Data on ILDOT Chart

Due to several borderline cases for rubblization and the fact that DCP testing coverage was limited, efforts were made to use the FWD data (taken at many more test locations) to better partition the project into sections where rubblization can be expected to succeed. To accomplish this segmenting, the ILDOT chart was referenced and it was predicted that a minimum subgrade CBR of 6 was necessary for the 7-inch thick PCC to be rubblized. Next, a relationship was evaluated between the FWD backcalculated subgrade modulus and the average DCP CBR of the first 12 inches beneath the slab. A plot of DCP CBR versus FWD modulus using all seven data points provides a relationship that is poor (low R^2 value) and shows a CBR value of 6 equates to a subgrade modulus of 7.3 ksi. If two outliers are eliminated (Locations 2 and 4), a better fit is found ($R^2 = .85$) as shown in Figure 3.5 that shows a CBR value of 6 equating to a subgrade modulus of 8.4 ksi. Outliers should not be judged with the sole purpose of improving a statistical fit. Rather, reason and logic must always apply in order to determine whether the available data set can provide a useful relationship. In this case, since the values 7.3 and 8.4 are reasonably close to one another, it was deemed that 8 ksi was a reasonable threshold for the minimum subgrade modulus necessary to achieve effective rubblization. This also roughly matches the correlation often used in pavement design of 1500xCBR = modulus (psi), or (1500)x6 = 9,000 psi (or 9 ksi).
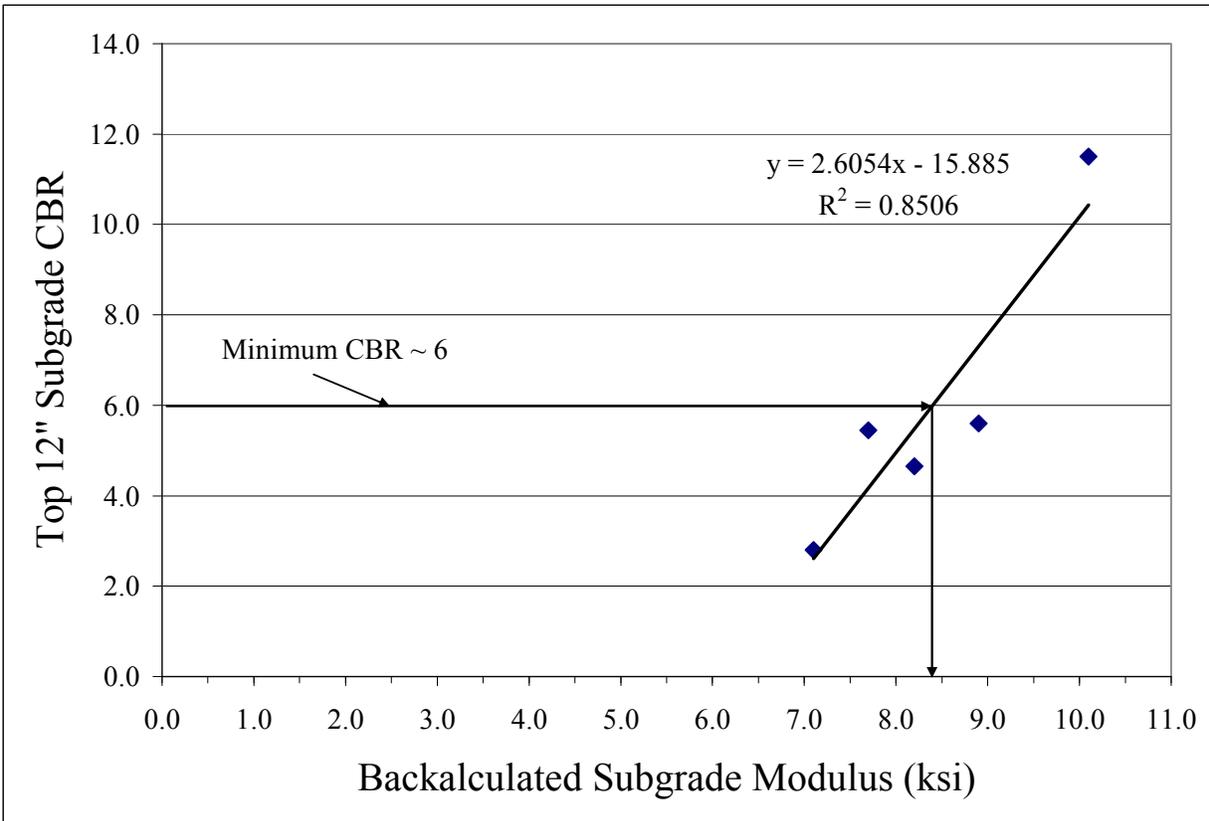
Figure 3.5, Subgrade CBR vs Modulus for Texas FM 1155

The backcalculated subgrade modulus determined from FWD testing was then plotted along with DCP test locations versus test station on the project, as shown in Figure 3.6. Segments were identified as "high risk" where the subgrade modulus is well below the 8 ksi threshold for at least two consecutive test stations (spaced every 250 ft) and as "medium risk" where the modulus is close to the threshold for more than one consecutive test station. For FM 1155, one "high risk" area around the 5000 ft station and two "medium risk" areas around the 500 ft and 7000 ft areas were identified as shown in Figure 3.6.
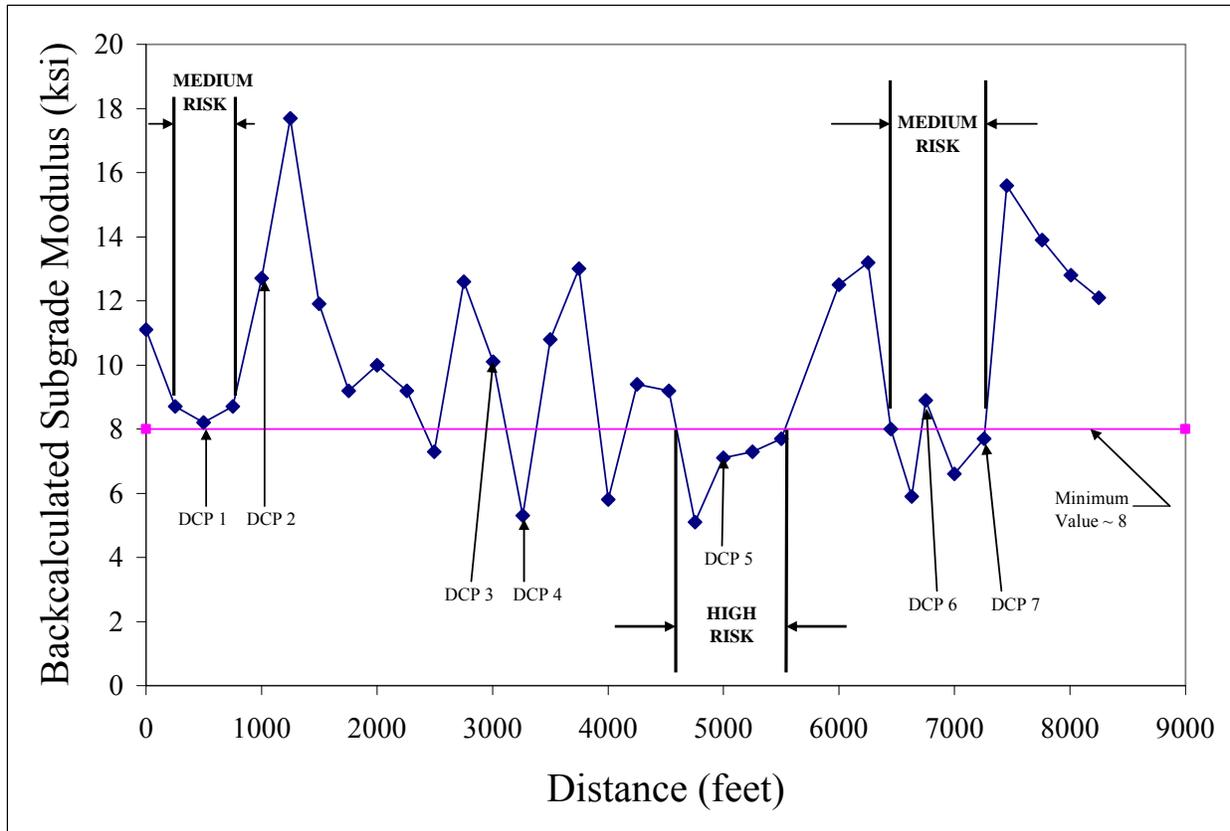
Figure 3.6, Identifying the Unstable Areas and Relative Risk on Texas FM 1155

At the time of finalizing this report, this project was designed for rubblization but not yet constructed. Because of the poor and variable strength conditions below the slabs on this project, TxDOT has specified four inches of crushed aggregate base be placed over the fractured concrete before the HMA overlay.

On a very limited number of projects at the time of this writing, TxDOT has been relatively happy with the effectiveness of this methodology in predicting sections that can be successfully rubblized versus those that cannot and require full-depth replacement.

3.2.5   Recommended Approach for Airfields

Using a pavement investigation plan similar to the TxDOT protocol allows the engineer to make well-informed decisions about whether a pavement is suitable for rubblization. By having an understanding of the amount, length and relative risk severity of these weak areas, the project engineer can estimate how much full-depth repair will be required, and even judge if rubblization is a feasible option for the project. If the decision is made to rubblize, it is very beneficial to know the areas that are most likely to cause problems so that all precautionary means can be implemented. Depending on what stability problems have been encountered early in the project and the relative support conditions of those completed areas, a more educated projection can be made of future difficulties for the remainder of the project. Even though the TxDOT protocol described above was developed for highway PCC pavements without stabilized

bases, it has features that can adopted to evaluate airfield pavements. For instance, the practice of utilizing a FWD and a DCP to evaluate the support conditions directly beneath the concrete is well recognized in the airfield pavement engineering community.

In developing an evaluation protocol for the wide range of pavement conditions found on airfields, we believe the ILDOT rubblization selection chart utilized within the TxDOT protocol should be adjusted based on recent experiences in Texas and other logical simplifications. The ILDOT criterion allows the MHB while eliminating the RPB for certain conditions. After reviewing all the national literature, project history and demonstration projects as part of this research, there is no compelling evidence to suggest that one type of equipment should be specified over the other for marginal support conditions. Both the RPB and MHB have experienced problems from rubblizing thin pavements with no base and a weak subgrade. Each equipment type has advantages and disadvantages, and some of the recommendations in Section 3.3 address equipment considerations. Regarding Method II of the ILDOT chart, it is not likely that RMI would try to operate their RPB without high-flotation tires on a pavement with questionable stability, especially as they have gained experience in this area.

Proof-rolling these marginally stable pavement sections after the fracturing process is very important, especially when using the MHB, because it operates on unbroken PCC while the RPB operates with two wheels on fractured PCC. Problems were encountered on a rubblization project on Texas SH 70 when weak areas were not detected during rubblization with the MHB, but were later discovered during paving operations when trucks and pavers caused excessive distortion and shifting of the large broken PCC pieces. No proof rolling occurred. Figure 3.7 is a photograph and illustration depicting this situation. Finding weak areas during the rubblization process or during proof rolling is much preferred over finding them later during paving operations when plants are producing and trucks are delivering HMA. Adjusting the rubblization effort to match in-place conditions, and the importance of proof rolling marginally stable sections, is further addressed in Section 3.3.3. This is the approach taken in Iowa and Wisconsin where many such pavements have been rubblized and overlaid with HMA and are performing very well.

Figure 3.7, Shifting of Rubblized PCC from Paving Operations on Texas SH 70

This research team developed the proposed criteria in Figure 3.8 for assessing airfields suspected of marginal support conditions. This new assessment chart, derived from the ILDOT chart, includes risk levels. The high risk zone is indicative of areas likely to be problematic, the moderate risk zone of potentially problematic areas, and low risk zone of areas that should rubblize nicely.

For areas slightly off the chart, engineering judgment should be used. For instance, if the PCC plus base thickness is 6-8 inches, and CBRs are 10-14, we believe the appropriate risk level is moderate. It should be noted that the Wisconsin criteria presented earlier in this chapter indicates that no "remedial action" is required if the subgrade has a CBR of 8 or greater. For PCC slabs 9 inches or greater, the literature and project history (chapter 2) suggests there will not be stability problems encountered while rubblizing, even if the subgrade CBR is very low. In these extremely rare cases, the PCC should provide adequate strength to support the rubblization process.

Unfortunately, there is little actual field data to validate the accuracy and recommended application of this chart. As with any new criteria, these guidelines need to be validated and possibly adjusted with actual field project experience. This validation can only occur with data collected from an evaluation framework similar to that provided by the TxDOT protocol and explained in the remainder of this section.
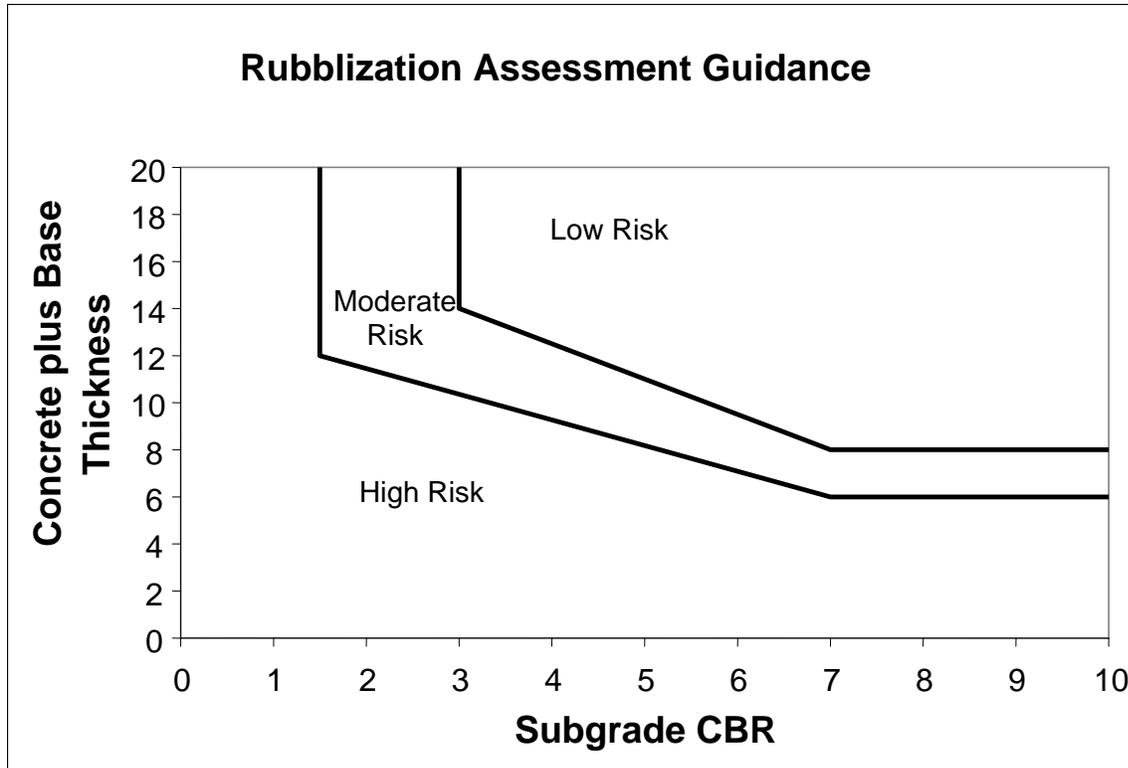
Figure 3.8, Proposed Rubblization Assessment Chart for Airfields with Slabs < 9 Inches

To evaluate airfield rubblization candidates where PCC slabs are less than 9 inches thick and the support conditions are suspect, it is recommended to adopt the evaluation framework provided by the TxDOT protocol in conjunction with the modified assessment chart in Figure 3.8. This will involve assembling typical section details of slab thicknesses, steel reinforcement, base layer and subgrade support conditions. A GPR survey is optional, but can help identify unknown section breaks as well as subsurface moisture problems, which can be a huge factor in affecting the ability to rubblize. If GPR is used, an air coupled survey is recommended for pavements thinner than 10 inches while a ground coupled survey is recommended for thicker pavements. An FWD survey is very important, and FAA Advisory Circular 150/5370-11A provides guidance in terms of testing location and frequency. Recommended is a minimum of two passes, each directly off each side of the centerline in the wheel-path with test spacing being 100-ft. For wide taxiways and on smaller runways, two additional passes are recommended, each 25 to 30-ft off the centerline with test spacing being 200-ft. For runways 150-ft wide, two more passes are suggested, each 50 to 65-ft off the centerline with test spacing being 200 to 400-ft. Coring and DCP testing should be conducted based on groupings identified from the FWD survey, with at least five test locations in areas of highest outer sensor deflections and five test locations from intermediate and low deflection areas. Additional tests may be very beneficial depending on the consistency of the FWD data.

In line with the TxDOT protocol, the risk assessment should be made at each test location utilizing Figure 3.8. While an evaluation could be done with only coring and DCP testing, this would be labor and time intensive and provide a less than comprehensive view of project conditions. By establishing a correlation between FWD subgrade modulus and DCP CBR (as the TXDOT protocol provides), a risk profile can be established over the entire project.

The frequency of FWD and DCP testing, and overall robustness of the evaluation will depend on a number of factors, including the level of concern based on current knowledge of in-situ conditions. Airfield pavements with a good quality granular or stabilized base will likely fall in the low risk area and are good rubblization candidates. On the other hand, thin slabs with no base and weak or saturated subgrade conditions will likely fall into the moderate or high risk area. This section provided the framework for assessing that risk, allowing the design engineer to predict the amount and location of full-depth repairs before the project starts, and once the rubblization process has started to adjust that prediction for the remainder for the project based on actual ability to rubblize under the known conditions.

3.3   Other Recommendations on Marginal Pavements

For a project with relatively thin PCC (< 9 inches) and suspect support conditions (thin to no subbase and weak subgrade), rubblization may still be the most attractive and economical rehabilitation alternative. If so, there are a number of recommendations and considerations in the planning, design and construction stages to compliment the assessment protocol and risk profile described in Section 3.2. While some of these are discussed and cross-referenced in other sections of this report, they are included here for convenience and completeness.

3.3.1   Install Edge Drain System

Trapped water directly beneath the slabs will inhibit the rubblization process. Installing longitudinal edge drains is recommended for all rubblization projects, unless a well-functioning drain system exists or the subgrade consists of a self-draining material such as sand. While edge drains are not always installed on highway and airfield projects, they will ease the rubblization process and will improve the overall long-term pavement performance. A review of current rubblization specifications was provided in Section 2.5.4–Drainage System, while further guidance and design details are in Chapter 6.

Ideally, the drainage system should be installed before the evaluation process described in Section 3.2 is conducted in order that any trapped water has time to escape. This may not be practical as installing drains prior to the evaluation indicates some degree of commitment to rubblize or at least to rehabilitate. The drains should be functioning at least two weeks (see Section 2.5.4) before rubblization is evaluated, or the test strip started, or a feasibility trial (as described in the next section) is conducted. This will allow the subgrade to dry out and better reflect the conditions that are to be expected during rubblization.

3.3.2   Conduct a Feasibility Trial

When the risk level has been assessed to be moderate or high on a large portion of the project (Section 3.2), a feasibility trial should be considered as was done on the Tullahoma runway project (discussed in Section 2.4) if the pavement is not operational. In the trial at Tullahoma, both rubblization contractors (RMI and Antigo Inc) were invited to demonstrate whether their equipment could effectively rubblize the project pavement before the final design was completed. If such a trial is done, the project evaluation data, analysis and risk assessment described in Section 3.2 should be shared with both contractors prior to the trial. Trial locations

should be selected based on the risk assessment, or at least on any evaluation testing that was conducted, in order to cover the full range of support conditions that exist. Test locations need to be identified in pairs that have similar conditions so that both equipment types can be evaluated on a level playing field. If the pavement cannot be effectively rubblized, using a "modified rubblization" technique or designing as "crack and seat" (which was done for the Tullahoma project) may still be attractive alternatives. Both of these fractured slab techniques are covered in upcoming sections as alternatives if the pavement is not able to support rubblization.

Such a feasibility trial is practical if the pavement is not operational, such as was the case for the Tullahoma project. If the project pavement is operational, however, down time and repair considerations make such a trial difficult unless there is part of the pavement that could be closed for an extended period. It may still be feasible to evaluate both equipment types at the start-up of a project. To guarantee such a trial, the contract documents would need to include this. As mentioned in the last section, the ideal timing of a feasibility trial is after the drainage system is installed and the subgrade has had time to reach an equilibrium condition. While a feasibility trial, whether done during the design stage or as part of the construction contract, will add additional expense and may be difficult to coordinate, the benefits are great on moderate and high-risk projects to avoid project delays and cost overruns during the rubblization process. On the other hand, if the risk assessment level is low for the majority of the project, a feasibility trial is not warranted.

3.3.3 Allow for "Modified Rubblization"

On marginally stable and thin PCCP, the MHB offers the advantage of being able to lower the hammer drop height and separate the impacts (by increasing the speed) to allow the PCCP to be broken in a less aggressive manner that produces a more suitable surface for an HMA overlay. This "modified rubblization" technique with the MHB typically results in fractured concrete pieces larger than the sizing criteria normally allowed for rubblization, but still can provide full-depth fracture through the slab and a surface suitable for an HMA overlay. Full depth fracture must still be confirmed with a test pit. The Wisconsin DOT will evaluate test strips and determine the appropriate amount of breakage for that project, or section of a project, with those marginal conditions. On a few projects, the Wisconsin DOT had determined in the design phase that the support conditions were not adequate for full rubblization and used a "modified break and seat" specification calling for the MHB to "break" the PCCP into 12 to 24-inch particles in order to maintain more of the PCCP's original support for the HMA overlay.

The feasibility trial at Tullahoma (section 2.4) illustrated this technique. Figure 3.9 shows the broken PCC at Tullahoma when the MHB first started rubblizing, with the normal 24-inch drop height and normal impact spacing. Since it was clearly apparent that these large PCC pieces were too disoriented to be able to be rolled into an acceptable surface for an HMA overlay, Antigo adjusted their MHB by lowering the drop height and increasing the impact spacing. Figure 3.10 shows the surface at Tullahoma after adjusting the MHB to a 16-inch drop height and 10-inch impact spacing, and after several passes of the Z-grid roller. Figure 3.11 shows the test pit from the same pavement in Figure 3.10, verifying full-depth fracture and revealing the particle sizing, which exceeded the specified criteria. As Figures 3.10 and 3.11 show, the resulting surface and fracture conditions of this "modified rubblization" technique appear to be suitable for an HMA overlay. The engineers on site at Tullahoma agreed.

Figure 3.9, Unacceptable Rubblization from Start-up of MHB at Tullahoma

Figure 3.10, Result of "Modified Rubblization" Process after Z-grid at Tullahoma

Figure 3.11, Test Pits from "Modified Rubblization" at Tullahoma

   This "modified rubblization" technique (with a MHB) is also referred to by Antigo as "aggressive crack and seat" because it resembles the conventional crack and seat process which utilizes a guillotine hammer, but provides more impact points and greater fracture (8-12 inch nominal size pieces) compared to conventional crack and seat (2-6 feet pieces). The conventional crack and seat process with a guillotine hammer is also an alternative on these marginally stable PCC pavements and is described next in Section 3.3.4.

   "Modified rubblization" should only be used when it has been deemed that the thin slabs and weak support conditions are preventing the specified size criteria from being met. Typically, the criteria for the top of the slab are most difficult to meet under these marginal support conditions versus the criteria for the bottom of the slab. Allowing larger pieces than allowed in most rubblization specifications can be warranted when a crushed aggregate base layer is to be placed between the rubblized layer and the HMA. Designing for an aggregate base layer placed over the rubblized surface is covered in Section 3.3.6.

   Each of the roller passes normally recommended when rubblizing with the MHB are still critically important: Z-grid roller, pneumatic rubber tire roller, and steel wheel roller. The Z-grid roller will further fracture surface pieces, the pneumatic roller will in-part serve as a proof-roller, and the final steel wheel will help provide a final smooth surface for the overlay. The pneumatic roller step is critically important on these relatively thin and marginally stable PCC pavements in order to identify any weak spots and ensure the PCC pieces are well seated and stable enough to support construction traffic for the ensuing overlay and the operational traffic for the new pavement design. While a 20 to 25-ton pneumatic rubber tire roller is typically specified, a lighter roller (such as 10-ton) may be considered for these thin PCC slabs. This is an engineering

judgment decision, as specifying too heavy of rollers and/or too many roller passes runs the risk of finding and replacing "weak areas" that may still be strong enough to support the construction and design traffic. On the other hand, too small of rollers and/or too few of passes may not find weak areas that will later fail. While the Z-grid and final steel wheel rollers are normally done with a vibratory roller, consideration should be given to running these passes in the static mode when the subgrade is saturated in order to not pull excess water up to the surface and create an even weaker condition.

Further proof rolling with a pneumatic rubber-tire roller, as recommended for the crack and seat method, may also be considered to further ensure the PCC pieces are well seated and stable enough to support ensuing construction traffic. Proof-rolling guidance for the crack and seat method on marginally stable pavements is provided in the next section, but the reader should understand that this guidance is for crack and seated PCC pieces that are larger than rubblized pieces and thus will support heavier loads. Rolling operations as part of the rubblization process, and proof-rolling, are discussed further in Ch 6 of this report.

3.3.4   Consider Changing Design to Crack and Seat

While the "modified rubblization" technique appeared to be suitable during the feasibility trial at Tullahoma, the engineer and owner for that project decided to redesign using conventional crack and seat. The crack and seat method utilizes a guillotine hammer to reduce the effective slab length of a PCC pavement by breaking it into smaller-sized pieces, ranging from 1 x 2 ft to 4 x 6 ft, with the most generally accepted dimensions being 1.5 x 3 ft. Since the PCC pieces are larger than with rubblization, the resulting modulus is generally higher and a new thickness design may be appropriate. On the Tullahoma project, a new design was accomplished using an assumed modulus for the crack and seat layer of 150,000 psi [Hall, 2007].

Crack and seat utilizes a guillotine hammer weighing up to 6 tons with a strike bar 4 to 6 inches thick and 5 to 10 feet wide, as seen in Figure 3.12. The hammer drop height is adjusted to ensure it provides enough energy to achieve full depth and full width fracture, but also that it minimizes spalling at the strike points. Impact spacing generally ranges between 18 to 36 inches, and is adjusted during the test strip operation to produce a desired breaking pattern in the concrete that will support the ensuing construction equipment, provide a modulus to support the structural design, and produce a surface that can be overlaid.

Rolling and seating of the fractured pavement is generally accomplished with two to five passes of a 50-ton pneumatic roller, although for thin pavements on a weak subgrade, more (3 to 7) passes with a lighter (30 to 35 ton) pneumatic roller is probably a more viable alternative. Note that the size of the pneumatic rollers for crack and seat pavements is larger than rubblized pavement, as the larger size PCC pieces can carry more load. The purpose of proof rolling "crack and seated" concrete pavement is to ensure full contact with the supporting subbase or subgrade layer, eliminate any rocking of the panels and to establish a firm foundation for the overlay. Steel wheel static or vibratory rollers are generally not satisfactory for seating the cracked pavement because they tend to bridge over the high spots on the broken PCC surface. For thin slabs, too heavy of rollers or too many passes could damage the subgrade. Again, engineering judgment comes into play on how heavy of rollers to use.

Figure 3.12, Guillotine Hammer Used with Traditional Crack and Seat Method

Crack and seat was a common rehabilitation method before rubblization became popular, and should remain as a viable fractured slab technology on plain jointed (no reinforcing steel) PCC pavements with thin slabs and poor support that may not support rubblization. Note that a similar technique called "break and seat" is utilized when reinforcing steel is in the PCC because the steel must be fully debonded from the PCC in order to eliminate unwanted slab action. Guidance and sample specifications on the crack and seat method are available in Chapters 10 and 13 of the Asphalt Institute MS-17 manual [AI, 1999] and some state DOT websites [N.Y. Eng. Instr., 1999] [N.Y. Spec., 1996]. An excellent paper on using the crack and seat method for rehabilitating airfield pavements was written in 1999 by Roy McQueen [McQueen, 1999].

3.3.5   Comparing "Modified Rubblization" versus "Crack and Seat"

The "modified rubblization" process with the MHB and "crack and seat" process with the guillotine hammer each seem to offer distinct advantages over the other. The "modified rubblization" process offers better control and more consistent breakage than the crack and seat process. If the PCC surface is rough, the wide guillotine hammer may only impact the PCC at high spots while the MHB has impact points across the entire machine's width. The MHB can avoid splinter breaks and damaging adjacent unrubblized pavement (isolated by full-depth saw-cut) better than the guillotine hammer. In addition, the modified rubblization process fractures the PCCP into smaller pieces compared to the crack and seat process, minimizing any possibility of reflective cracking. On the other hand, the crack and seat process with resulting larger PCC pieces may provide a more stable platform to support the ensuing construction overlay traffic and

the design aircraft traffic. The larger PCC pieces should also provide a higher modulus of the fractured PCC layer, resulting in a thinner HMA overlay design thickness requirement. In addition, many companies have a guillotine hammer for crack and seat while only Antigo has the MHB equipment for "modified rubblization," which should result in a slightly lower cost to fracture the PCC.

3.3.6   Include an Aggregate Leveling Course

Designing the new pavement section with a crushed aggregate base leveling course, such as FAA P-209 material, directly on the rubblized surface will minimize any difficulties with the rubblized surface being too rough or uneven. In addition to providing a smooth and dense surface for the HMA overlay, the aggregate layer should also act as a crack relief layer. This will provide additional insurance that no reflective cracking will occur, especially if the rubblized pieces are larger than normally specified due to thin slabs on weak support conditions. Airfield projects have also utilized crushed concrete or reclaimed asphalt pavement (RAP) for a leveling course. While a leveling course will help protect the rubblized and underlying layers, heavy haul trucks, grading and compaction equipment must still deliver, place and compact the aggregate. The haul trucks may pose a critical loading period for the exposed rubblized surface. Further guidance on placing an aggregate leveling course over rubblized material is provided in Ch 6.

3.3.7   Include Separate Bid Item for Full Depth Patching

Providing an estimated quantity and separate bid item for full depth repairs will help ensure a competitive price for this work. In addition, costly delays and overruns will be minimized, as the contractor will be anticipating these failed areas and have the necessary equipment and material on hand to repair. Based on discussions with select state DOTs, a reasonable planning estimate for full-depth repair is 0-10% for pavements assessed as "low risk," 10-25% for pavements assessed as "moderate risk," and greater than 25% for pavements assessed as "high risk." A review of current rubblization specifications was provided in Section 2.5.4–Unstable Area Patching, while further guidance is in Chapter 6. Photographs of full depth patching are in Figures 2.9, 2.10 and 2.12.

3.3.8   Other Modifications during Construction

The following recommendations or considerations are modifications to different aspects of the construction process that may increase the probability of being able to successfully rubblize thin PCC pavements with poor support conditions.

Avoid "Wet Season" when Rubblizing

In line with installing effective drainage, the seasonal timing of the rubblization process can be critical to ensuring water is not trapped under the PCC, which can inhibit the rubblization process. Scheduling the project to avoid a wet season, such as Spring, can make a big difference in whether the thin PCC pavement can be successfully rubblized or not. Both the MHB and RPB contractors we spoke to emphasized the importance of this item, and stated that conditions can

go from acceptable to unacceptable in a matter of weeks depending on the rain and moisture conditions underneath the slabs. It is never a good idea to let a rubblized pavement surface be exposed to precipitation for any significant period of time since water will penetrate easily.

Turn Vibrators Off on Rollers

Both rubblization contractors agree that when rubblizing under wet subgrade conditions, or when the water table is within 5 feet of the surface, strong consideration should be given to turning the vibrators off during the roller operation immediately after the PCC has been fractured. Subgrade moisture will have a tendency to migrate up toward the bottom of the fractured slabs with heavy and repeated loadings, especially when these loads are vibrating. The multiple stages of the rubblization process where this can occur are the passes of the MHB or RPB, Z-grid, pneumatic and/or steel wheel rollers, HMA haul trucks, HMA paver and HMA compaction equipment. Some engineers have theorized that the many vibratory operations with rubblization, combined with a sand/silty subgrade close to saturated conditions, can lead to excessive pore pressures and a liquefaction state [Mehta, 2002]. Running the roller operations (that are part of the normal rubblization process) in static mode versus vibratory mode can help increase the chance of success with rubblizing marginally stable pavements.

No HMA Belly Dump Trailers and Windrows

In some areas of the United States, HMA is routinely placed using a technique called windrow paving. Belly-dump trailers are used to lay out the HMA in a longitudinal windrow directly in front of the paver, and a machine then picks up the HMA and places it in the paver hopper. The advantage is that the paver does not have to stop for trucks unloading. This technique is not desirable on any rubblized layer because the loose PCC pieces, which exist on the surface with a lack of fines, can be picked up rather easily and pulled into the HMA. On thin PCC slabs that have been rubblized, this is especially a problem because the belly-dump HMA delivery trucks are extremely heavy and can cause the larger rubblized pieces to dislodge and overturn. If the HMA pick-up equipment catches the edge of a large PCC piece, it can cause dislodging and overturning of that piece. See the illustration and photo in Figure 3.7 where this occurred on a project in Texas.

High Flotation Tires with RPB

When rubblizing with the RPB, the high-flotation (low pressure) tires should be utilized. These were developed specifically for the RPB to avoid rutting on thin rubblized slabs. While the rutting is less likely with the high-flotation tires, it can still occur as has been shown in Figures 2.8 (Pratt Runway project) and 2.11 (Kegelman Runway project) and is illustrated in Figure 3.13 (Texas highway). As discussed earlier, the RPB can identify unstable areas because one side operates on rubblized material. It is desirable to identify any "weak areas" during rubblization or proof rolling and not during paving operations when HMA is being produced and delivered to the jobsite.
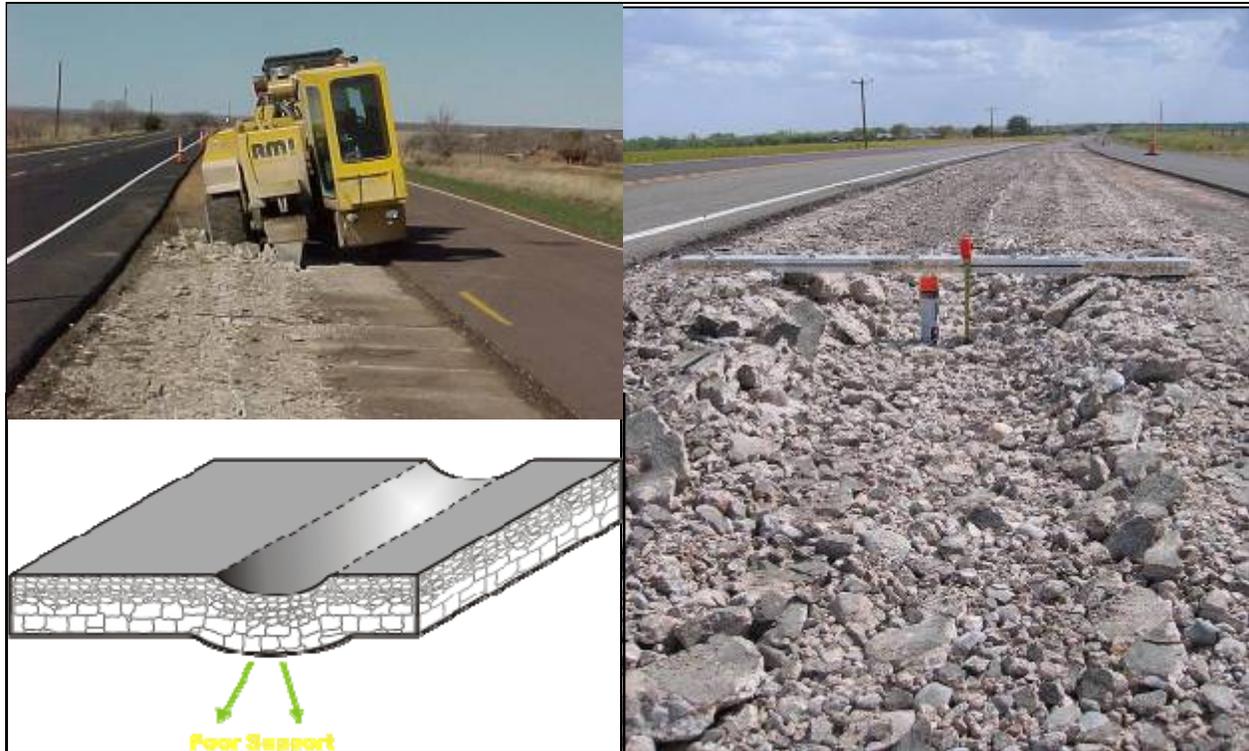
Figure 3.13, RPB Causing Rutting Failures on a Texas Highway

Tracked HMA Pavers on First Lift

The HMA pavers on tracks distribute the heavy HMA load better than pavers on wheels, making them desirable on thin PCC that has been rubblized. In Illinois, it was reported on I-57 that a tracked paver disturbed the loose rubblized surface much less than a rubber-tired paver [Heckel for IL DOT, 2002]. They also provide better traction on the loose rubblized surface, which is desirable when placing the first lift of HMA.

Avoid Trafficking HMA Trucks and MTVs on Rubblized Surface

On an airfield project, it may be possible to sequence the rubblization and the first lift paving operations to where heavy HMA haul trucks and material transfer vehicles (MTVs) can operate on adjacent lanes that have not been rubblized in delivering the HMA to the paver. While MTVs are good to minimize segregation of the HMA and to allow continuous paving operations without stopping, they are also very heavy and should be used with extreme caution when on relatively thin PCCP that has been rubblized. The use of MTVs may actually assist in keeping heavy HMA haul trucks off the rubblized pavements by operating the MTV on adjacent intact PCCP and transferring the HMA over to the paver on the rubblized.

CHAPTER 4 – MATERIAL CHARACTERIZATION AND OTHER THICKNESS DESIGN CONSIDERATIONS

This chapter covers thickness design considerations for rubblized PCCP. Prior to discussing how to characterize the rubblized layer in a structural thickness design, an overview of airfield pavement design methodologies is provided. Both the layered elastic and CBR design procedures are covered because both are currently used in designing rubblized airfield pavements. Also included in this overview is a summary of the current design guidance in FAA EB-66 [FAA, 2004]. A large portion of the chapter is then spent summarizing backcalculated moduli values of rubblized PCCP layers from projects found in the literature (performed by others) and also backcalculated moduli calculated by our research team as part of this project. These values are summarized to develop a range of recommended design modulus values. Two other research concepts for characterizing a rubblized layer are then explored utilizing the data already presented: 1) a "retained modulus" approach and 2) modeling the rubblized material as two layers. Both concepts need additional data for further validation before they can be recommended for implementation. For utilizing the CBR design procedure, "equivalency factors" are suggested for characterizing a rubblized layer. Minimum HMA overlay thickness criteria is discussed by considering practical issues such as compaction, smoothness and profile. Finally, the results of a sensitivity analysis are presented to illustrate the impact that different rubblized moduli values have on the required HMA overlay design thickness.

4.1   Overview of Airfield Pavement Design Methodologies

4.1.1   Introduction

FAA thickness design procedures are included in FAA Advisory Circular (AC) 150/5320-6D, "Airport Pavement Design and Evaluation." For flexible pavement designs, which are appropriate for asphalt overlays on rubblized concrete, the FAA permits either the California Bearing Ratio (CBR) method described in Chapter 3 of FAA AC 150/5320-6D, or the mechanistic-empirical layered elastic design method described in Chapter 7 of the same AC. Computerized versions of the CBR and layered elastic design procedure are embodied in FAA's F806FAA and LEDFAA Programs, respectively.

For new generation aircraft such as the B-777 and A-380, the layered elastic design methods must be used since CBR design charts are not included in the same Advisory Circular for these aircraft. For fleet mixes that do not include these aircraft, either the CBR or the layered elastic method can be used for the thickness design. It is our understanding that the FAA will archive the CBR design method upon adoption of their new FAARFIELD design procedure, which will utilize the the current layered elastic methods for flexible pavement design in LEDFAA. FAARFIELD is scheduled for final release in 2007.

Pavement thickness design procedures for U.S. military airfields are encased in the PCASE suite of software programs. Both the CBR method and a layered elastic procedure referred to as LEEP can be used, and are relatively similar to the FAA design procedures. Since we are not aware of any plans for the military to archive their CBR design procedure, the CBR design methodology is recognized and addressed in this report.

When utilizing fractured slab technologies (crack/break & seat or rubblization), pavements are designed as a flexible pavement system because the rigid PCCP slabs are essentially converted into a flexible behaving layer. For crack & seat (on PCCP with no reinforcing steel) and/or break & seat (on PCCP with reinforcing steel), the slabs are fractured full depth by a mobile drop hammer with specified crack patterns generally 1 x 2 ft to 4 x 6 ft. Rubblization is performed with either a MHB or a RPB, also fracturing the slabs full depth but into much smaller pieces, generally less than 1 ft maximum. The extent of fracture significantly affects how the fractured concrete layer will behave and thus how it should be characterized. As the extent of fracture increases, the modulus will decrease along with the structural capacity of that pavement layer. On the other hand, as the extent of fracture decreases, the modulus and structural capacity increases. The greater the modulus value assigned to the rubblized layer, the thinner the HMA overlay design thickness will be. While there is structural benefit to attaining a stiffer rubblized layer, concrete that is not sufficiently fractured can cause a greater likelihood of reflective cracking into the asphalt overlay.

4.1.2 CBR Design Method

Current FAA and military CBR design methods utilize the familiar CBR design equation to protect successive unbound base/subbase and subgrade layers from shear failure [Ahlvin, 1991]. The CBR equation was modified in the 1970's to include alpha factors to adjust flexible pavement thickness designs for different aircraft gear configurations. The total pavement thickness (i.e., surface, base, subbase) is based on a fixed asphalt surface thickness on an aggregate base (CBR typically 80 or 100) on a user-defined CBR subbase on a user defined CBR subgrade.

FAA design procedures require the use of a stabilized base and subbase when designing new pavements for aircraft fleet mixes that include gross loads in excess of 100,000 lbs. These stabilized layers are handled with equivalency factors. For a stabilized base material, the equivalency factors range from 1.2 to 1.6, meaning 1.0 inch of stabilized base is "equivalent" to 1.2 inches to 1.6 inches of unbound crushed aggregate (FAA P-209 material with CBR = 80). For subbase materials, the equivalency factors range from 1.0 to 2.3, meaning 1.0 inch of stabilized subbase is equivalent to 1.0 to 2.3 inches of unbounded aggregate subbase (FAA P-154 material with CBR = 20). Guidance and ranges on choosing an appropriate equivalency factor for stabilized layers is provided in FAA AC 150/5320-6D, Section 321.

Traffic in the CBR method is handled by equating the mixed aircraft fleet to a design aircraft using the logarithmic relationship contained in Chapter 3 of the same AC. The design aircraft is defined as the aircraft in the fleet mix giving the thickest pavement at its weight and departure level.

For all the past projects examined in this research project designed by the CBR method, the rubblized layer was characterized as a crushed stone aggregate base.

4.1.3 Layered Elastic Design Method

As with most mechanistic design methods, pavement layers are defined by the elastic modulus, Poisson's ratio, thickness, and layer interface (all layers assumed to be fully bonded for flexible pavements). For flexible pavements, FAA and military layered elastic design procedures are based on computing vertical strains at the top of the subgrade for a layered pavement. A

supplemental design criterion based on horizontal strain at the bottom of the asphalt layer is also available in LEDFAA, but will rarely govern given the default asphalt modulus and minimum layer thicknesses used for design. The subgrade or asphalt strains are computed by FAA's LEAF layered elastic computational program [Hayhoe, 2002]. As with the CBR method, the failure criteria is based on the results of full scale tests to failure, with primary failure defined as one-inch upheaval in the sub-grade (shear).

The LEDFAA program will either use default moduli, or allow the user to select a modulus within a pre-defined range for stabilized base or subbase. The stabilized base moduli ranges were established during calibration studies to yield thicknesses similar to those obtained from the CBR method for the equivalency factor ranges embedded in that procedure. Moduli for unbound aggregate base (P-209) and unbound subbase (P-154) are automatically computed as a function of the layer thickness and modulus of the underlying layer [Barker et al, 1975]. LEDFAA permits an allowable range of modulus values for subgrades of 1,000 to 50,000 psi.

A cumulative damage approach is used in LEDFAA with elastic strains computed for each aircraft utilized in the fleet mix. An aircraft wander pattern is also assumed and strain damage is accumulated according to Miner's hypothesis across the pavement. A terminal condition is reached when the cumulative damage factor (CDF) equals 1.0, with the CDF defined as the summation of actual number of load repetitions (from the forecast) for each aircraft divided by the allowable number of load repetitions (from the strain criteria) for each aircraft. More detail on the CDF and flexible pavement failure criteria can be found in the LEDFAA user's manual (help file).

Historically in the LEDFAA program, rubblized layers have typically been characterized as an unbound crushed stone material, an equal substitute for P-209 material.

4.1.4   Thickness Design Guidance in FAA EB-66

The design guidance within EB-66 states that either the CBR method (Chapter 3) or the layered elastic design method (Chapter 7) of FAA AC 150/5320-6D may be used in determining the required HMA overlay thickness. When using the CBR method, the guidance states, "most rubblized material will perform equal to or better than P-209 material. Unless additional project specific information is available, a one-to-one substitution should be used in the design procedures provided that sufficient subgrade conditions exist to allow proper rubblization."

For the layered elastic method, EB-66 references the guidance in AI's MS-17 manual. Specifically, a design modulus for the rubblized layer with a 95% reliability factor should be selected. This is equal to the average modulus value (from data) less one standard deviation. Since designs are typically performed without any rubblized field data, EB-66 goes on to say, "rubblized pavement moduli have been found to vary from a low of 30 ksi to over 300 ksi, depending on slab thickness, base type and condition of base layers. Pavements that are less than 9 inches and have marginal bases and subgrade conditions will be at the low end of the modulus range. Thicker pavements over 9 inches with good bases or stabilized bases have shown much higher modulus values. Pavements constructed by pre WW II methods typically have low modulus values." EB-66 goes on to say larger size fractions and any bonded steel reinforcement will produce higher modulus, but if too high then the possibility of reflective cracking may exist.

4.2   Average Modulus Values from other Agencies

This section provides a summary of results from searching the literature for backcalculated modulus values of rubblized layers on past projects. Some of these references are also discussed in Chapter 2 of this report, but this section focuses on the backcalculation and characterization of the rubblized layer. Backcalculated moduli for the subgrade are also included in this summary when available in the literature, mainly to examine if there is a significant change in values between pre and post rubblization.

4.2.1   Witczak and Rada Paper

This 1992 TRB paper [Witczak et al, 1992] reported on the first nationwide performance study and comparison of the various fractured slab technologies. This study was the same as the one referenced in Chapter 2 by PCS/Law, Inc for NAPA [PCS/Law, 1991]. Figure 4.1 from the study compares the average section moduli and their variability associated with Rubblized, Crack and Seat (C&S) and Break and Seat (B&S) layers.
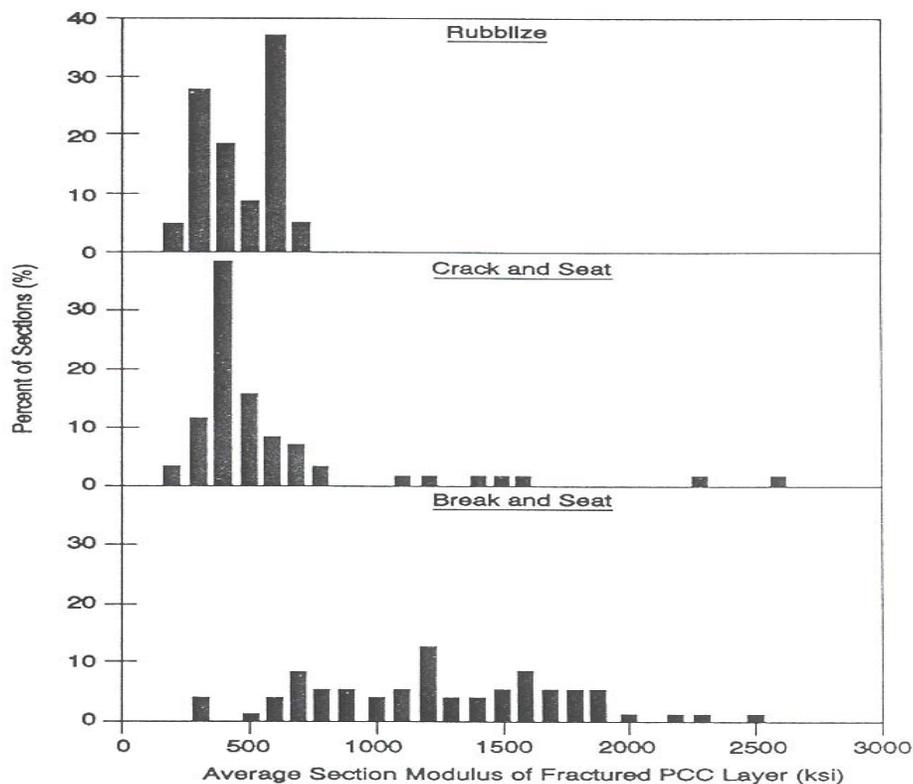
Figure 4.1, Frequency Distribution of In-situ PCC Moduli after Fracture [Witczak, 1992]

For the 22 rubblization sections examined, the backcalculated moduli ranged from 200 to 700 ksi, with a mean value of 412 ksi and standard deviation of 154 ksi (CV of 37%). For 46 C&S sections with a moduli under 1000 ksi, the average moduli was 409 ksi with a standard

deviation of 141 ksi. There were also 18 additional C&S sections that the authors considered outliers with moduli over 1000 ksi. Combining all 64 C&S sections, the average moduli was 780 ksi with a standard deviation of 666 ksi. For the 52 B&S sections included in the study, the average moduli was 1,271 ksi with a standard deviation of 549 ksi. Comparatively between the three processes, both the average moduli and the variability was highest for the B&S sections, followed by the C&S sections, and lowest for the rubblized sections. Intuitiveley, this is logical because particle size is much larger with C&S and B&S relative to the rubblization, and the reinforcing steel inherent with B&S sections should also increase stiffness. It also is logical that the variability of B&S sections is the highest because of the variable degrees of debonding that likely exist between the steel and the PCC. The average modulus of 412 ksi for the 22 rubblized sections is high relative to the other studies that will be covered in this chapter. This may be due to the fact that most of these pavements were rubblized in the mid-1980s as the process was just being developed. Improvements in equipment and experience over time may have led to lower typical moduli for rubblized. Witczak also introduced the concept of retained modulus ($E_{rubblized}$ / $E_{unbroken\ pcc}$). For the rubblized sections, he reported retained modulus values between 8% and 10% respectively. This concept is explored later in this chapter.

4.2.2 Selfridge ANG Base Runway

As covered in Chapter 2, a paper was presented on the 2002 project to rubblize the Selfridge ANG Base runway [Anderson, 2004]. The existing PCC runway had extensive D-cracking and maintenance was continually needed to control FOD potential. The slabs thicknesses ranged from 13 to 21 inches and rested on a layer of medium fine sand between 3 and 5 ft thick. The subgrade support was very weak with design CBR's between 1 and 3. The water table was reported to be only 3 feet below the runway surface, and in some cases only 12 inches below the bottom of the PCC. Edge drains were retrofitted into the pavement prior to rubblization. The MHB was used to rubblize. For thickness design purposes, the CBR for the rubblized section was set at 100.

HWD deflection data were collected to evaluate and compare the moduli of the rubblized layer and of the subgrade before and after rubblization. The results are shown in Table 4.1.

Stations	Concrete thickness (inches)	Existing		After Rubblization	
		Concrete Modulus, x 10^6 (psi) Avg/StdDev	Subgrade Modulus (psi) Avg/StdDev	Rubblized PCC Modulus, x 10^5 (psi) Avg/StdDev	Subgrade Modulus (psi) Avg/StdDev
117+50-127+00	16	4.37 / 2.62	18,648 / 3,021	4.30 / 1.58	18,019 / 1,670
127+00-160+00	21	2.73 / 2.28	21,819 / 8,890	3.70 / 1.96	23,368 / 3,545
160+00 – 170+00	13	2.05 / 0.57	14,615 / 2,791	1.83 / 3.77	16,251 / 1,240
Combined	---	3.07 / 1.19	19,486 / 7,301	3.49 /1.87	21,290 / 4,247

Table 4.1, Pre and Post Moduli for Concrete and Subgrade at Selfridge [Anderson, 2004]

Anderson concluded that "it is evident that the rubblized layer is an order of magnitude less than the concrete, namely 180,000 to 430,000 psi as compared to 3 million psi for the concrete. For comparison, a typical high quality crushed aggregate base has a modulus of 30,000 psi, and hot mix asphalt of about 400,000 psi." Some notes of interest on the reported results of the Selfridge project are:

- Although conditions existed for possible build up in pore water pressure (shallow water table and sand subgrade as reported by FL DOT), no stability problems were reported during construction. The extremely soft subgrade (CBRs of 1 to 3) also did not apparently inhibit the rubblization process. Field performance has been outstanding.
- This study is one of the few where moduli are reported before and after rubblization. The results in Table 4.1 are converted to a "retained modulus" in Figure 4.2. Note the thicker slabs provide the higher retained modulus. The mode of fracture explained in the Michigan study [Baladi, 2002] discussed in Chapter 2 can explain this. On thicker slabs, a greater portion of the bottom slab thickness contains only hair-line fractures with a high degree of angular interlock versus the "fluff" layer that occurs at the top.
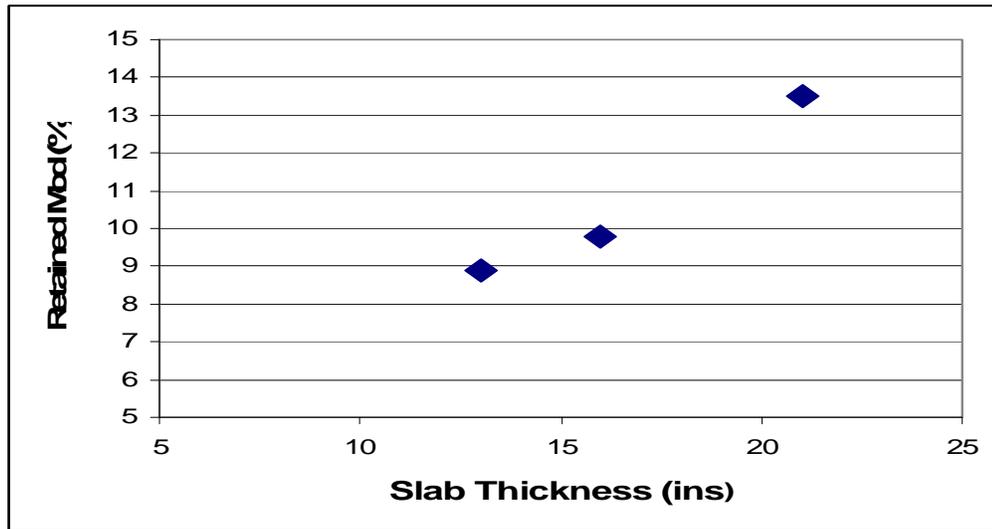
Figure 4.2, Retained Modulus versus Slab Thickness at Selfridge ANGB

- The average subgrade moduli in two of the three sections had a slight increase, although considering the reported standard deviations, this increase is not significant. Some engineers might have anticipated a drop in backcalculated subgrade moduli, depending on how the pavement profile was modeled with a subbase, because of an expected increased deviator stress after rubblization. This was not the case.

4.2.3　US Army Corps of Engineers Report

The COEs had been actively monitoring three rubblization projects: Selfridge ANGB runway, Hunter Army Airfield taxiway and Niagara Falls Air Reserve Station runway. These results were published in a paper [Velez-Vega, 2005] discussed in Chapter 2. Table 4.2 is a summary of their HWD results. Several comments regarding these results are:

- For the Selfridge project, the reported moduli are substantially higher than those reported by Anderson (same project) as discussed above. Anderson reported moduli from 183 to 430 ksi, whereas Velez-Vega reported values from 741 to 2,700 ksi (see Subbase layer in Table 4.2). A layer of crushed recycled PCC was placed above the rubblized layer, and reported moduli values for this layer were from 77 to 129 ksi (see base layer in Table 4.2). It is unclear from this paper when (year and month) the FWD testing was done. The upper end (2,700 ksi) is similar to the unbroken concrete modulus reported by Anderson. No explanation is available for this large difference. The assumptions used were not reported and the raw deflection bowls are not available for further analysis.

- For the Hunter project, Antigo claims it was Crack and Seat and not rubblization as reported by Velez-Vega. The high modulus value for a 6-inch slab supports Antigo's claim.

- For the Niagara Falls project, the reported average moduli for a 9.5 inch slab ranged from 101 to 156 ksi, depending on the assumption made regarding depth to a stiff layer. The report also states that there was a shallow water table, no construction problems were encountered and all the monitored sections are performing very well.

Facility Information			Layer Information							
			Pavement Overlay		Base		Subbase		Subgrade	
Military Installation/Location	Rubblized Date	Feature	Thickness mm (in)	Back-calculated Modulus MPa (psi)	Thickness mm (in)	Back-calculated Modulus MPa (psi)	Thickness mm (in)	Back-calculated Modulus MPa (psi)	Thickness mm (in)	Back-calculated Modulus MPa (psi)
Selfridge Air National Guard Base / Michigan	Summer 2002	Runway 01-19 (4 different Rubblized sections)	180 (7.0) AC O/L	1,760 (255,735)	114 (4.5) Crushed PCC (leveling course)	890 (128,730)	483 (19.0) Rubblized PCC	5,110 (741,430)	1,270 (50.0) Poorly Graded Silty Sand (SP-SM)	150 (21,720)
			180 (7.0) AC O/L	1,760 (255,735)	114 (4.5) Crushed PCC (leveling course)	575 (83,450)	406 (16.0) Rubblized PCC	12,800 (1,870,000)	254 (10.0) Fine Coarse Sand SM	160 (22,820)
			180 (7.0) AC O/L	1,760 (255,735)	114 (4.5) Crushed PCC (leveling course)	740 (107,260)	533 (21.0) Rubblized PCC	8700 (1,260,900)	1473 (58.0) Medium Sand SP	140 (20,380)
			180 (7.0) AC O/L	1,760 (255,735)	114 (4.5) Crushed PCC (leveling course)	530 (77,320)	330 (13.0) Rubblized PCC	19,200 (2,784,875)	330 (13.0) Poorly Graded Silty Sand (SP-SM)	145 (21,050)
Hunter Army Airfield / Savannah, Georgia	2003	East Taxilane	152 (6.0) AC O/L	1,260 (182,480)	152 (6.0) Rubblized PCC	4,070 (590,750)	N/A	N/A	Poorly graded Sand (SP)	130 (18,930)
Niagara Falls Air Reserve Station, New York	Unknown	Runway 10L / 28R (Data available for 2 different Rubblized sections)	130 (5.0) AC O/L	1,760 (255,735) Based on Temp	240 (9.5) Rubblized PCC	1,080 (156,930)	330 (13.0) Silty Sand Gravel	30 (4,450)	Silty Gravelly Sand (Comp Subgrade)	210 (30,735)
		Comments: The PCASE results show a shallow depth to bedrock. These results are based on the FWD data provided, further analysis should be made to consider depth to bedrock and subgrade strength. The subgrade strength changes considerably from stations.	130 (5.0) AC O/L	1,760 (255,735) Based on Temp	240 (9.5) Rubblized PCC	700 (101,090)	330 (13.0) Silty Sand Gravel	68 (9,802)	Silty Gravelly Sand (Comp Subgrade)	460 (66,530)

Table 4.2, COE Results from Rubblized Sections on Three Airfields [Velez-Vega, 2005]

4.2.4   Colorado DOT I-76 Project

In 1999, the Colorado DOT rubblized a portion of I-76 with both the RPB and MHB equipment. As discussed in Chapter 2, the long-term performance of this project was reported in a report [LaForce for CDOT, 2006]. The existing pavement was 8 inches of JPCP (no steel) and had substantial ASR damage. The existing thin overlay was removed, edge drains installed, the PCC was rubblized and 6 inches of HMA was placed on top. Both the East Bound (EB) and West Bound (WB) sections were reported to be performing well in 2006. FWD testing was done before and after rubblization in 1999 and again in 2004. Backcalculation of the "after rubblized" subgrade resilient modulus and an effective resilient modulus are reported in Table 4.3.

	EB Resonant Breaker	EB Multi-head Hammer	WB Resonant Breaker	WB Multi-head Hammer
1999 Subgrade Resilient Modulus	16,374	18,224	19,991	17,354
2004 Subgrade Resilient Modulus	16,373	19,672	19,776	16,525
1999 Effective Pavement Modulus	86,926	61,481	79,665	99,195
2004 Effective Pavement Modulus	318,158	293,381	251,458	248,651

Table 4.3, Colorado's FWD Results [LaForce for CDOT, 2006]

The results in Table 4.3 show both the subgrade modulus and the "Effective Pavement Modulus" computed with the Darwin design program both immediately after rubblization (1999) and 5 years later (2004). The "Effective Pavement Modulus" cannot be compared with other E_{rubb} values in this report because it is a composite modulus of all layers above the subgrade. Comments by LaForce regarding these results are:

- No differences were reported between the subgrade moduli calculated before rubblization, immediately after rubblization, and over 5 years later. The subgrade was permeable sand.
- The type of rubblization equipment (MHB versus RPB) used did not seem to be a factor in the subgrade modulus or effective pavement modulus.
- There was a dramatic increase in calculated "Effective Pavement Moduli" from 1999 to 2004. LaForce believed this was due to a combination of: 1) cementing of the rubblized concrete and 2) stiffening of the HMA layers. Although no FWD results or test conditions were given in the report, Laforce commented that the average FWD deflections decreased from a range of 15-19 mils to 7-8 mils, indicating substantial stiffening over time.

The results from this Colorado project are also significant because severe ASR damage was the primary distress in the existing concrete. After 5 years of service, both sections were reported to be performing very well (see ASR, Chapter 6).

4.2.5   Illinois DOT I-57 Project

In 1990 the Illinois DOT rubblized a section of I-57 near Pesotum. Details can be found in a TRB paper [Thompson, 1999]. The original section was 10 inches of JRCP over 6 inches of granular base. After rubblization, two different HMA overlay thicknesses were used (6 and 8 inches). FWD data was collected on an annual basis for 7 years and the results of the backcalculation were summarized. From data presented in this paper, Figure 4.3 was developed showing the increase in backcalculated moduli (from WESDEF) for the 8-inch thick HMA section.
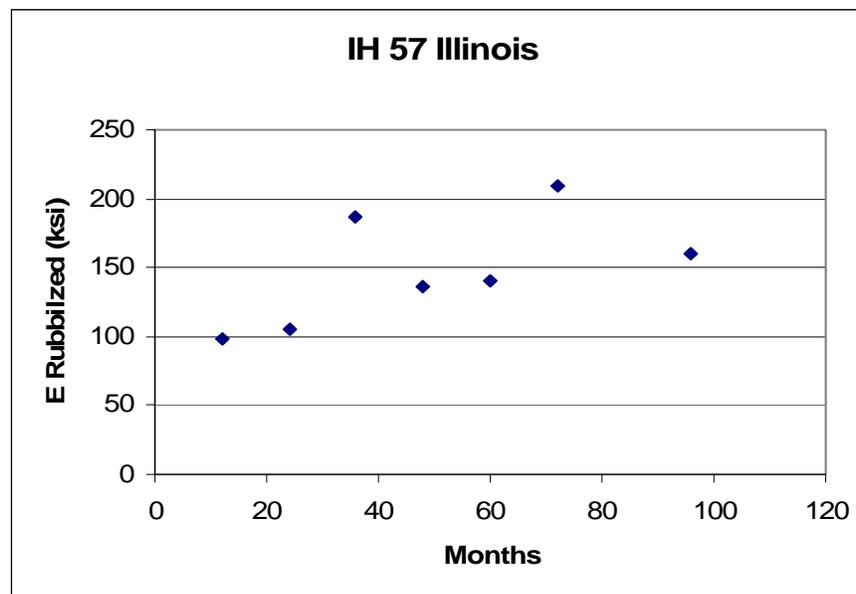
Figure 4.3, Change in Rubblized Modulus with Time, I-57 Illinois

The cause of the increase in modulus of the rubblized layer with time is not known. One possible cause could be re-cementing of the fractured concrete particles. From Thompson's work, the average modulus calculated for all sections was reported to be134 ksi, substantially higher than granular base material.

4.2.6   Michigan DOT Report

The most complete set of deflection data available was assembled and processed in the MI study [Baladi et al, 2002]. This study, on the performance of rubblized pavements in Michigan, has been summarized in Chapter 2. In Phase II of this study, they conducted field and laboratory testing as well as backcalculation and mechanistic modeling. The pavements in Michigan are largely 9 inch JRCP with temperature steel at mid-depth. They concluded that the concrete in these sections was not rubblized uniformly with depth. Above the temperature steel

the pavement was broken into material with a composition similar to granular base. They referred to this top layer as "rubblized" material, but henceforth in this report we will refer to it as "crushed" material. Below the steel, the material was classified as fractured concrete, retaining a fractured slab structure. Based on this, they recommended two methods for processing the FWD data. In the first, the average modulus of the entire rubblized layer was computed. In the second, the rubblized layer was divided into two layers and an estimate was made of the stiffness of each layer. This was not done through normal backcalculation, as it is very difficult to model two relatively thin layers (4.5 inches each) with backcalculation and get reasonable results. Instead, an analysis procedure was developed using the deflection bowls and a set of regression equations developed by the researchers. This method is explained in the report. As an example, one section had a single backcalculated modulus for the 9 inch slab reported to be 122 ksi, but using the regression equations this was equivalent to 4.5 inches of modulus 65 ksi and 4.5 inches of modulus 256 ksi.

Based on the summary results from 18 sections in Michigan, the average values reported are shown in Table 4.4.

Material	Thickness (ins)	Average Modulus (ksi)	Range of Modulus (ksi)
Combined Rubblized Layer	9	207	46-1064
Top of Rubblized Layer (crushed)	4.5	87	12-297
Bottom of Rubblized Layer (fractured)	4.5	503	92-2774

Table 4.4, Average Moduli Reported in Michigan Study

In addition, the researchers studied the design moduli of granular base materials used in Michigan and concluded that the backcalculated modulus was twice as high as those used in the pavement design process. The authors stated that if the "crushed" concrete (top of rubblized layer) was equivalent to granular base material, then it was justifiable to reduce the average modulus (87ksi) of that layer by a factor of 2 for design purposes. Their final recommendations were to use the following moduli for design:
- Combined Rubblized Layer   207 ksi
- Top of Rubblized Layer (crushed)   43 ksi
- Bottom of Rubblized Layer (fractured)   503 ksi

The decision on whether to model the Rubblized layer as one or two layers has a major impact on the mechanistic design computations, particularly the tensile stains computed at the bottom of the HMA layer. This will be discussed later in Section 4.5.2.

One consequence of this study was that MDOT tightened its specifications and field inspection on future rubblization projects. They currently inspect the breakage pattern thoroughly and insist that the reinforcing steel be 100% debonded. This often means that the rubblization equipment must make additional passes over the same area. Concerns have been raised by some rubblization contractors that this may have an overall detrimental effect to the

pavement system since the "crushed" layer in some cases is claimed to be "powdered." MDOT currently has a task force looking into this issue and their rubblization specification.

Applied Pavement Technology Inc later conducted a separate long-term performance study on the initial MI projects using the MHB included in the Baladi 2002 study. This study, mentioned in Chapter 2 and summarized in a TRB paper [Wolters et al, 2007], found that there was no correlation between the reported breakage pattern by Baladi and the ultimate long-term pavement performance. The study concluded that the eventual long-term performance was more related to the presence of under-drains than the extent of breakage. HMA overlay thicknesses were between 5.5 to 10.5 inches, with the average about 7.5 to 8.0 inches.

4.2.7   Indiana DOT US 41

Indiana DOT efforts to obtain layer modulus values on two rubblized sections of US 41 were reported at TRB [Gala, 1999]. The first section was 8 inches of CRCP and the second was 10 inches of JRCP. The CRCP has considerably more reinforcing steel compared to the JRCP. The pre-rubblized moduli were 3882 and 3692 ksi, respectively. After rubblization, the reported average moduli were 181 ksi for the 8-inch CRCP section (standard deviation of 54 ksi) and 166 ksi for the 10-inch JRCP section (SD of 28 ksi). Thus, the pre- to post-rubblized modulus value ratios are similar.

4.2.8   Detroit Metropolitan Airport Trial

As discussed in Chapter 2, two short trial sections at the Detroit Metropolitan Airport were rubblized in 2006 to evaluate the RPB and MHB equipment for rehabilitating the runway. Both sections consisted of 17-inches of PCC with wire reinforcement (4 inches below surface) on top of 8 to 9 inches of HMA base. After rubblization, a 3-inch HMA overlay was placed only to provide a surface for HWD testing and not to support traffic. The construction of these test sections were documented in an internal report [Decker, 2006] while the HWD results are summarized in a letter report [Kohn, 2006].

For the RPB section, the average backcalculated modulus was 229 ksi with a standard deviation (SD) of 110 ksi. For the MHB section, the average was 113 ksi with a SD of 42 ksi. These moduli values are low compared to those reported from other projects (i.e. Selfridge AFB) with similarly thick slabs. Our own backcalculation of the FWD data verified similar low moduli for these 17-inch thick slab sections.

As reported in Section 2.4 and by Decker, the test pits showed that the MHB section had broken full depth while the RPB section did not, possibly accounting for the higher modulus values and higher variability of the RPB relative to the MHB. Still, the average RPB modulus of 229 ksi is surprisingly low considering the PCC was not fractured full-depth (at least in the test pits). Antigo reported (in e-mail correspondence) that they erred on the side of more breakage rather than less in order to ensure full-depth breakage of the 17-inch thick PCC on a 9-inch HMA base. They did this by preceding the MHB with a 6-ton guillotine hammer with closer strike spacing than is typically used on pavements of similar thicknesses without an HMA base. This offers a possible explanation of why the backcalculated moduli for the MHB (average of 113 ksi) is relatively low compared to other projects documented in this chapter with similarly thick slabs.

Kohn reported a pre-rubblized PCC modulus of approximately 5500 ksi.

4.2.9   FAA's National Airport Pavement Test Facility (NAPTF)

McQueen provides a complete discussion of the testing, analysis and findings on these test sections rubblized in 2005 in Appendix A of this report. A brief summary of the test sections, trafficking and rubblization observations was provided in Section 2.4. This section summarizes those aspects that relate to the backcalculation efforts at NAPTF.

FWD testing was repeated at multiple drop weights and on two different dates prior to trafficking. McQueen used both the FAA's backcalculation method (BackFAA) as well as the Corp of Engineer's (WESDEF) method to calculate moduli for the rubblized layer and subgrade. Table 4.5 below was developed from the values shown in Table 7 of Appendix A to show average moduli for the MRC, MRG and MRS sections on both dates, and then an overall average.

	Pre-Traffic Average Back Calculated Moduli (ksi)					
	MRC		MRG		MRS	
Date	Rubblized	Subgrade	Rubblized	Subgrade	Rubblized	Subgrade
6/5	314	14.9	534	16.0	310	13.3
6/17	429	13.3	637	16.1	267	13.1
Overall Avg.	371	14.1	586	16.1	289	13.2

Table 4.5, Average Moduli for NAPTF Sections

The coefficient of variability (CV) of the moduli at individual stations reported in Appendix A is about 30%. The subgrade moduli in these sections did not change significantly from those reported before rubblization (Table 3 of Appendix A). The MRG subgrade moduli are higher than those from the MRC and MRS sections, as were the CBR readings, probably because of the moisture that allegedly drained down when the P-154 subbase was placed during construction of the MRC and MRS sections.

These average rubblized moduli are relatively high at NAPTF compared to those reported elsewhere. This may be explained by examining the rubble excavated from the test trenches (see Figure 4.4). It is apparent that the slabs were fractured full depth but not to the gradation limits typically required in rubblization specifications. The fractures were hairline with a high degree of particle interlock below the top 3 inches of "crushed" material. Personnel on-site estimated that about 80% of the steel dowels examined from the test pits remained bonded to the concrete.

Figure 4.4, Excavated Rubblized PCC in MRG Section at NAPTF [Garg ppt, 2006]

This illustrates one of the major questions regarding rubblization: "What level of breakage is enough to achieve the primary goals of a stable foundation that will not be a source of reflective cracking?" With the accelerated loading at NAPTF, the rubblized sections performed quite well under the initial 50 kip (4-wheel) loads. Failures developed only when the loading was increased to 65 kips (6-wheel). The failures were related to subgrade shear. It is very possible these sections would have failed earlier if the rubblized material had been finer with a corresponding lower modulus value.

Table 4.6 shows different ways the "retained modulus" percent can be calculated for the three sections, depending on back-calculation methods. Two "types" of average pre-rubblized PCC moduli are shown, one using only the "Closed Area" values reported in Table 3 of Appendix A and the other using the "Closed Area" and "Layered Elastic" values reported in the same table. The post-rubblized moduli in Table 4.6 are the averages from Table 4.5. Thus, two different "retained modulus" percents are calculated in Table 4.6 for each of the three sections.

	Average Pre-Traffic Back Calculated Moduli (ksi)			"Retained Modulus" %	
	Pre-Rubblized PCC Using Closed Area (CA) Values Only	Pre-Rubblized PCC Using CA and Layer Elastic (LE) Values	Post-Rubblized PCC (Back FAA and WESDEF)	Using CA Only	Using CA & LE
MRC	2,976	3,895	371	**12.4**	**9.5**
MRG	2,735	3,572	586	**21.4**	**16.4**
MRS	3,233	4,871	289	**8.9**	**5.9**

Table 4.6, "Retained Modulus" Percents for NAPTF Sections

4.2.10 Rubblization Project in Chile

Rubblization technology (using the RPB) has been available in Chile since 2004. A study of a 5 km pilot project found the backcalculated modulus value to be between 2.7 and 3 times greater than for an aggregate base with a CBR >80% [Thenoux et al, 2006].

4.3   Average Modulus Values from New Backcalculations

In conducting this project, our research team took several available FWD data sets from the projects listed below to further evaluate post-rubblized backcalculated moduli. These data were processed using the MODULUS 6 software [Liu, 2001] to develop new backcalculated modulus values of rubblized layers.

4.3.1   Texas DOT US 83

US 83 Highway in the Childress District consisted of a thickened edge 9-6-9 pavement and was rubblized in 2004 with the RPB. A 0.6-mile monitoring site was established and FWD data was collected before rubblization, and then 6 and 18 months after placement of the HMA surface. In performing the backcalculations, a uniform slab thickness of 8 inches was used and a depth to stiff layer was fixed at 130 inches. The results are shown in Table 7.

	Backcalculated Modulus	
	Intact or Rubblized PCC, (ksi) Avg/Std Dev	Subgrade, (ksi) Avg/Std Dev
Before Rubblization	3,072 / 950	10.8 / 1.6
6 months after Rubblizing	114 / 33	12.2 / 1.2
18 months after Rubblizing	199 / 60	12.8 / 1.5

Table 4.7, Backcalculated Moduli from US 83 Texas

From the data presented in Table 4.7:
- The rubblized layer modulus increased from 114 ksi to 199 ksi over 12 months.
- The sandy subgrade modulus did not show a significant change after rubblization.
- The "retained modulus" values after rubblization was 3.7% at 6 months and 6.5% at 18 months.

The rubblized section is performing very well. The average deflection after 18 months was 8.7 mils with a standard deviation of 0.39 mils (6.7%), which is considered very uniform.

4.3.2   Michigan DOT I-75

The MDOT report [Baladi, 2002] already discussed in this chapter and in Chapter 2 provides extensive FWD data on numerous rubblized sections in Michigan. Some conclusions were that the slab action was not always completely destroyed and the temperature steel was not always completely debonded. As a consequence, MDOT has tightened down enforcement of their specification that requires all steel to be 100% debonded. According to the rubblization contractors, this means additional passes of their equipment are required and the results are that the top layer can get "powdered." This additional effort may have detrimental impact on the structural capacity of the overall pavement system. This theory is supported by the analysis below.

Figure 4.5 shows four average FWD deflection bowls from different sections of I-75 in Michigan. In all cases, the sections were 9-inch thick rubblized slabs and the FWD testing was conducted on only the first lift of HMA (3.5 inches) soon after rubblization. The two deflection bowls labeled "2000" were taken before the year 2000 and are from the Baladi report. These are average bowls from 40 drops on the two original tests sections on I-75. These are compared with the two bowls labeled "2006", which were collected by MDOT personnel [Bancroft, 2006] from

a 2006 Michigan project on I-75, which was executed under MDOT's tighter enforcement of the rubblization specification. One of these "2006" lines is from the RPB while the other is from the MHB, but no significant difference between the equipment types is noted. Both, however, show significantly higher deflections on the inner sensors relative to the two "2000" deflection lines.
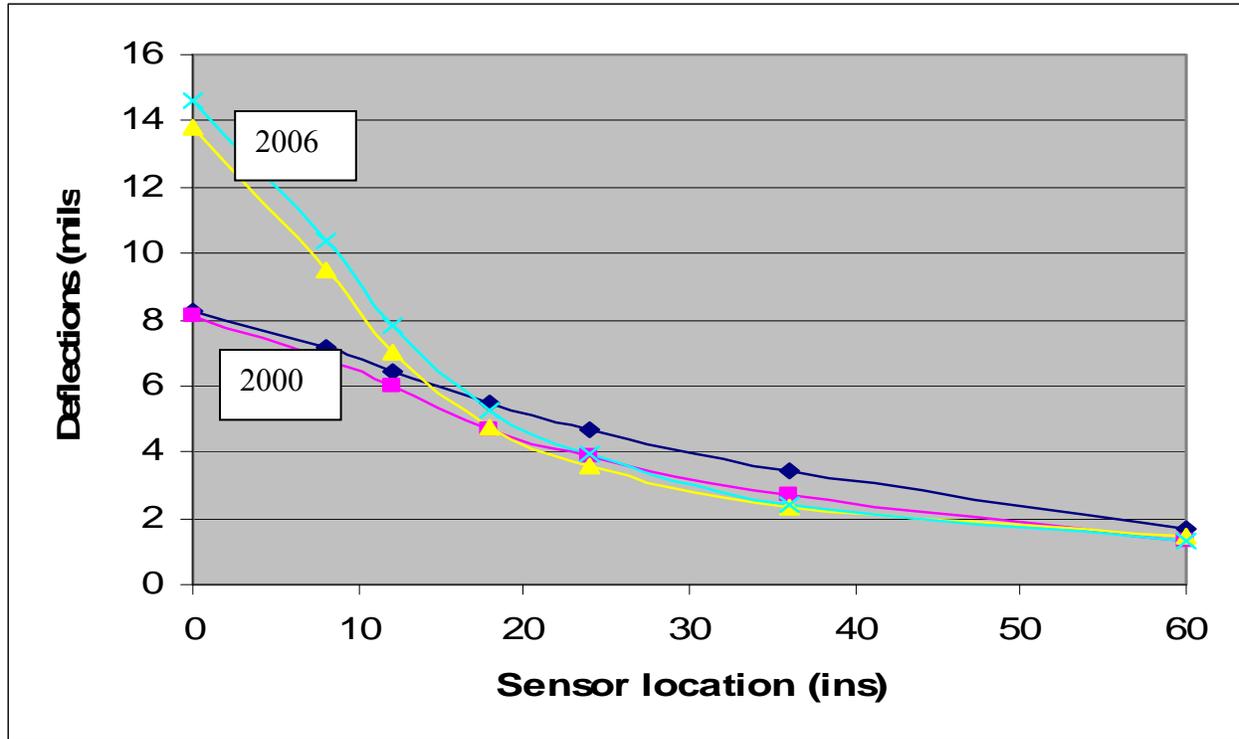
Figure 4.5, Average Deflection bowls from I-75 Michigan

The average backcalculated moduli of the rubblized layer from each of these sections are shown below in Table 4.8.

Year Section Was Built	I – 75 Site	Rubblized Layer Modulus (ksi)	Standard Dev (ksi)	Number of Deflections
Old (pre 2000)	Test site 1	225	70	40
Old (pre 2000)	Test Site 2	188	47	40
2005-2006	CS 06111 MHB	57	28	75
2006	CS 06111 RPB	53	12	55

Table 4.8, Moduli of Rubblized Layers on I-75

The results shown in Table 4.8 are an indication that substantially different moduli can be obtained depending on the level of rubblization energy applied. Repeated passes of the rubblizing equipment will cause additional fracture in the concrete, losing interlock and turning the slab into a material that is much closer to the equivalent of a granular base. The modulus

values of 57 and 53 ksi are approaching those expected for a granular base. How a specification is interpreted and enforced can vary significantly in the field and may contribute to differences in moduli. This has considerable implications on the fatigue life of the structure.

It is recognized that this analysis and the subsequent conclusions are considerably limited. First, this was not a controlled experiment and the sections may have had different support conditions. Second, the sections were different ages at the time of testing. The "2005" FWD data was collected shortly after rubblizing, while the "2000" data was taken from rubblized sections several years old. Several other studies have indicated that the rubblized stiffness can increase with time, which may have been a factor in the analysis.

4.3.3 LTPP Sites in Illinois

Rubblized pavements were included in the Special Pavements Studies of the LTPP program [Ambroz, 2005]. Six states constructed sections around 1990 and monitored the sections for over 10 years. Unfortunately, only two states (Illinois and Arizona) collected regular FWD data after construction. The FWD data collected on the Illinois sites (170663 and 170664) were extracted from the LTPP database and processed as part of this research project. The 170663 and 170664 sections were both 10" thick slabs, had edge drains retrofitted, and had HMA overlays placed with thicknesses of 8-inches and 6-inches, respectively. The concrete was rubblized with the RPB. Construction log comments stated the specification target was "no pieces to be larger than 6 inches."

The FWD data extracted from the LTPP database was collected before rubblization and at regular intervals afterwards. The data was processed by our project team using MODULUS 6.0 [Lui, 2001] and the depth to stiff layer was fixed at 20' below the surface. The maximum deflection data had a coefficient of variability ranging from 7% to 10% on each project, indicating uniform support to the HMA layer. Table 4.9 shows the average moduli at both sites. These averages were calculated from 26 to 30 deflection bowls per site. Note the values shown for 04-05-90 are prior to rubblization.

| Test Date, Condition | Backcalculated Modulus Values (Standard Deviation) Ksi | | | |
| | Site 170663 | | Site 170664 | |
	PCC	Subgrade	PCC	Subgrade
04-05-90 Intact PCC	5,975 (602)	16.5 (1.1)	5,618 (1,398)	15.1 (0.3)
11-16-90 Rubblized	146 (92)	17.5 (2.3)	100 (39)	16.2 (1.2)
08-05-93 Rubblized	112 (46)	18.3 (2.4)	89 (30)	16.7 (1.0)
06-28-95 Rubblized	140 (40)	19.5 (3.0)	107 (31)	18.1 (1.8)
10-17-97 Rubblized	236 (91)	20.5 (2.7)	142 (44)	19.1 (0.5)
10-08-98 Rubblized	233 (85)	20.2 (2.9)	184 (77)	19.7 (2.3)

Table 4.9, Moduli for Illinois LTPP Sites 170663 and 170664

The rubblized moduli are plotted versus time in Figure 4.6. For both sites, the modulus values are lowest around 3 years after rubblization and continue to rise in later years. It should be noted that the minimum values are still at least twice as large as commonly used moduli values for crushed stone aggregate base. "Retained modulus" values ranged from a low of 1.9% to a high of 3.9% for site 170663; and from a low of 1.6% to a high of 3.2% for site 170664. There seemed to be a slight increase in subgrade moduli over time on both projects, but this may be due to seasonal effects.
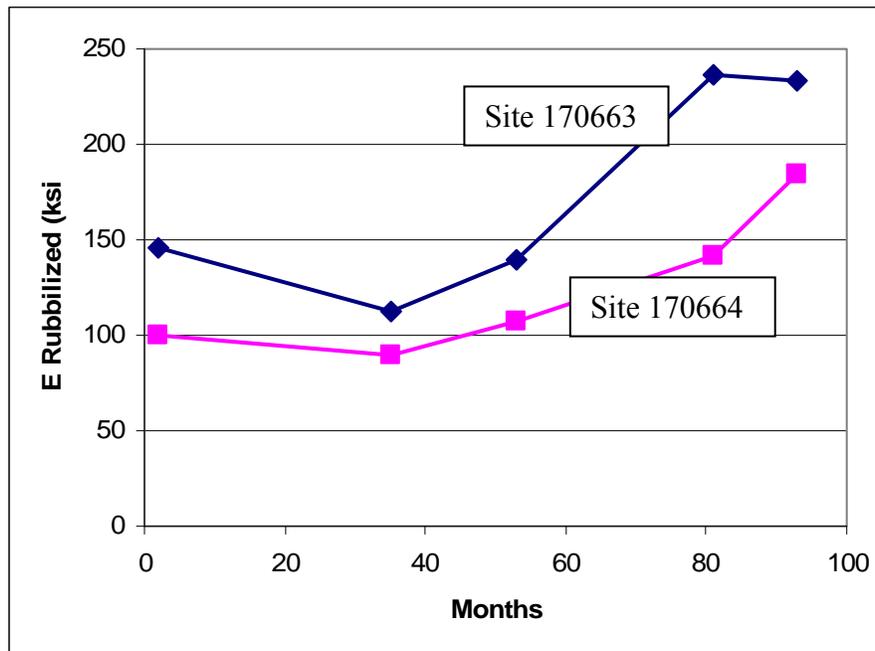
Figure 4.6, Plot of Rubblized Layer Moduli versus Time from Illinois LTPP Sites

4.4   Recommended Design Modulus Values

In selecting design modulus values of rubblized layers for future projects, it is clear from the discussion already presented that a range is appropriate given various factors such as slab thickness, the level of rubblization applied, the specification that is enforced, etc. At one end of the scale is the most recent data from Michigan indicating that repeated passes of the MHB or RPB can break the existing slabs down to a granular aggregate base. These sections resulted in moduli values in the 50 to 60 ksi range. At the opposite end of the scale are the results from the FAA's NAPTF, where large PCC pieces were interlocked and the dowel bars were often not debonded. These sections resulted in moduli values in the 300 to 600 ksi range. It is also apparent that there were no obvious trends or differences with regard to the two rubblization

equipment types, as the MHB and the RPB produced both higher and lower moduli at different times on similar sections, as well as moduli in the same range on other sections.

Slab thickness appears to be the clearest variable to examine with regards to helping predict an in-place modulus of the rubblized layer during the design phase of the project. Relationships to variables not known until rubblization has occurred and a test pit has been dug, such as some measure of rubblization effectiveness like average particle size, would not be helpful to the designer. Slab thickness to a certain degree appears to indirectly relate to two factors that intuitively should influence in-place modulus; particle size and degree of interlock. For thicker slabs, the rubblized particles tend to be larger and the interlock is generally stronger, leading to a higher modulus. For thinner slabs, which often are built on poor support, the unstable conditions may cause poor particle interlock leading to a lower modulus. While in-place modulus also appears to be related to the level of rubblization effectiveness and resulting particle size distribution, there is insufficient data at this time to develop methodology for selecting a design modulus of the rubblized layer based on specification gradation requirements. In addition, the specified criteria are not always an indication of what actually occurs in the field.

A summary of all the average initial backcalculated modulus values for rubblized layers in the pavement sections presented in Sections 4.2 and 4.3 are tabulated in Table 4.10, along with the appropriate slab thickness.

Reference (Site) *Authors: of this report	Location in This Report	Slab Thickness (inches)	Backcalculated Modulus (ksi)
Anderson (Selfridge)	Table 4.1	16	430
Anderson (Selfridge)	Table 4.1	21	370
Anderson (Selfridge)	Table 4.1	13	183
Velez-Vega (Niagara Falls)	Table 4.2	9.5	156
Velez-Vega (Niagara Falls)	Table 4.2	9.5	101
Thompson (IDOT)	in text	10	134
Galal (INDOT)	in text	8	181
Galal (INDOT)	in text	10	161
Kohn (Detroit Airport)	in text	17	229
Kohn (Detroit Airport)	in text	17	113
McQueen (NAPTF)	Table 4.5	12	371
McQueen (NAPTF)	Table 4.5	12	289
*Authors (US 83-Texas)	Table 4.7	8	114
*Authors (I75-Michigan)	Table 4.8	9	225
*Authors (I75-Michigan)	Table 4.8	9	188
*Authors (LTPP-Illinois)	Table 4.9	10	146
*Authors (LTPP-Illinois)	Table 4.9	10	100

Table 4.10, Summary of Initial Backcalculated Moduli of Rubblized Layers

For this data set, we decided to exclude two sections, a very high value and a very low value, that we felt were not representative of a "typical" rubblized section. The extreme low end value was the recent (2005) I-75 data from Michigan (Table 4.8). This decision was based on the presumption that these low values were a result of the additional passes required by MDOT

for both types of equipment to ensure 100% debonding of all steel. The top of these rubblized sections were reported to be "powderized" by the rubblization contractor. The extreme high end value was the MRG section at NAPTF (Table 4.5). For PCC slabs with thicknesses of 12 inches, building directly on grade (without a base or subbase) is not representative of past or present airfield construction, although the same could not be said of old thin slab airfields. Figure 4.4 (shown earlier) and Figure 4.7 (below) are photos showing the extremely large PCC pieces from MRG. It is possible that the lack of a base or subbase on MRG had a dampening effect with the RPB equipment which may have influenced particle size.

Figure 4.7, Photo of Large Concrete Piece from MRG at NAPTF

Therefore, the 17 data points in Table 4.10 represent moduli that could be anticipated with typical airport pavements and normal enforcement of typical rubblization specifications. The 17 data points in Table 4.10 are plotted in Figure 4.8 to show the relationship between modulus and slab thickness.
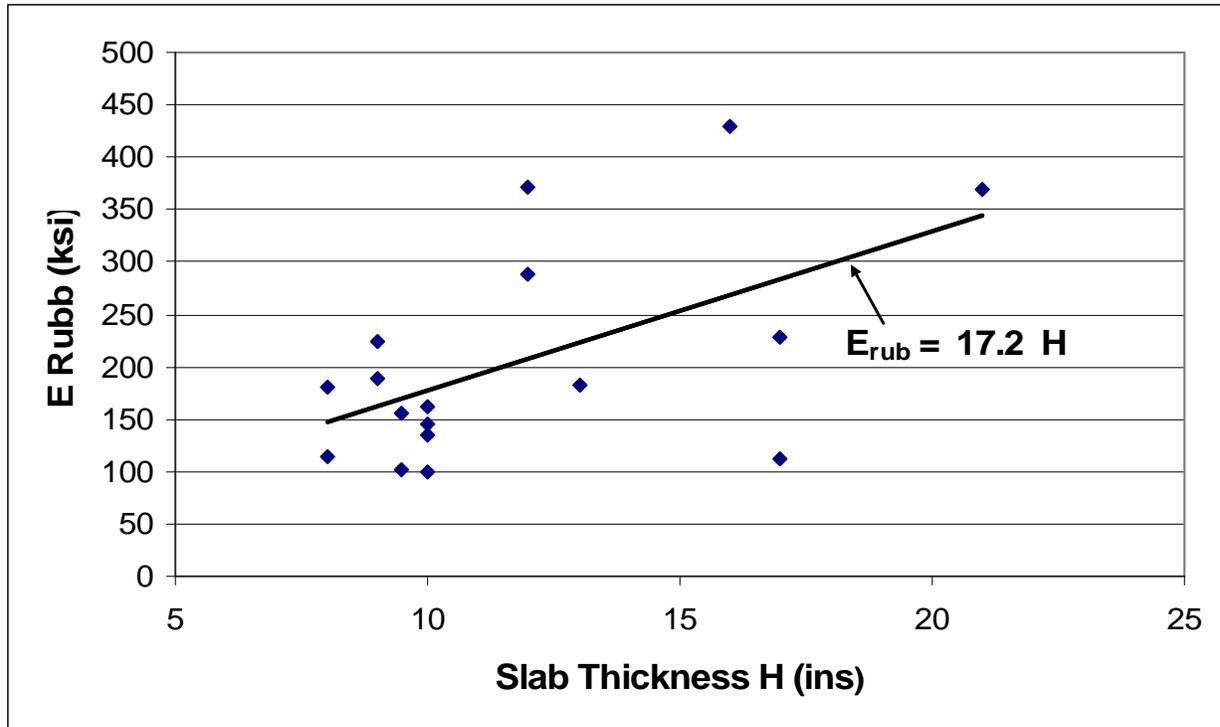
Figure 4.8, Average Initial Moduli versus Slab Thickness for All Sections Discussed

Using regression techniques, the following equation was developed:

$$E_{Rubb} = 17.2\ H \qquad\qquad R^2 = 0.32$$

where

E_{Rubb}: Backcalculated modulus of the rubblized concrete (ksi)

H: PCC slab thickness (inches)

It is recognized that this correlation ($R^2 = 0.32$) is poor, and would be worse if one or both of the extreme values were included. For example, including just the MRG value would lower the R^2 value to 0.22. On the other hand, not including the lowest value from Detroit AP (113 ksi with 17 inches) would raise the R^2 value to 0.5.

Besides this collection of backcalculated values, there are several published items in the literature that provide recommended moduli ranges for rubblized PCC. The first four items have been discussed and referenced in Chapter 2 and/or earlier in this chapter. These are summarized below.

- Witczak & Rada Paper [Witczak et al, 1992]: 200 to 700ksi with mean of 412 ksi
- Asphalt Institute's MS-17 Manual [AI, 1999]: at least 250 ksi typical
- FAA's EB-66 [FAA, 2004]: 30 to 300 ksi (low end of this range for thin slabs and high end of this range for thick slabs)
- PerRoad Users Guide [APA, 2006]: 300 to 700 ksi, 500 ksi typical
- AASHTO's New Mechanistic Empirical Pavement Design Guide [ARA, 2004]: 150 ksi

Comparing these recommended values with the collection of backcalculated values summarized in Table 4.10 and Figure 4.8, the recommendations in FAA's EB-66 and in the new

AASHTO M-E Design Guide are conservative relative to the others, but also the most accurate. The others appear to be slightly high relative to the findings here.

Based on the data and discussion presented thus far in this report, the following ranges are suggested for selecting a design modulus value of rubblized PCC on airfields:

- For slabs 6 to 8 inches thick: Moduli from 100 to 135 ksi
- For slabs 8 to 14 inches thick: Moduli from 135 to 235 ksi
- For slabs >14 inches thick: Moduli from 235 to 400 ksi

Considering the wide range of modulus values and the low R^2 value from Figure 4.8, the design engineer may choose to be more conservative and select a value on the low end of these ranges. The exact value selected may be influenced by considerations such as level of conservatism in the design, exact slab thickness within the above ranges, pre-rubblized PCC modulus (see next section), anticipated particle size and steel debonding conditions, and relevant historical data.

It is evident from the data presented in this chapter that the current design practice of characterizing rubblized PCC as a crushed stone base (50-60 ksi) is conservative. Since the range of moduli for rubblized PCC has been found to be 100-400 ksi, it may be more accurate to characterize rubblized PCC as a different material category with a modulus in the range of a flexible asphalt base. The moduli range for a flexible base in LEDFAA is 150-400 ksi. Characterizing as a flexible base versus a crushed stone base seems intuitively logical since a rubblized layer is a combination of "crushed" and "fractured" concrete.

The data in Table 4.10 and Figure 4.8 are initial modulus values. There were four studies cited earlier in this chapter (LaForce - CO, Thompson - IL, LTPP Sites in IL, US 83 in TX) whose data indicates some level of increase in the backcalculated moduli for a rubblized layer over time. Unsubstantiated theories for this increase include further compaction or some type of re-cementing that may occur within the rubblized PCC. Because standard design procedures do not allow for modulus value changes over time, and due to the limited nature of the data indicating this trend, our recommendation is that this not be a consideration when selecting a design modulus.

To summarize, the following conclusions are made based on the data, analysis and discussion presented thus far in this chapter.

- Moduli of in-service rubblized material was found to be in the 100-400 ksi range, significantly higher than that of crushed aggregate base with a typical range of 50-60 ksi.
- Rubblized modulus appears to be influenced by slab thickness, although there was a poor R^2 correlation factor from the data. Thicker slabs tended to have higher modulus. The following ranges are suggested for design on airfelds:
  - For slabs 6 to 8 inches thick: Moduli from 100 to 135 ksi
  - For slabs 8 to 14 inches thick: Moduli from 135 to 235 ksi
  - For slabs >14 inches thick: Moduli from 235 to 400 ksi
- There was an indication on four separate projects that the rubblized modulus increased with time.
- Rubblized modulus is dependent on the level (amount) of rubblization effort. Repeat runs of either rubblization equipment tend to reduce the final modulus.
- Heavily reinforced slabs can cause concern regarding effectiveness of rubblization, as there can be minimal breakage below the steel. That being said, there were no rubblized

projects found in the literature that have reflective cracking in the asphalt overlay caused from unfractured PCC slabs thought to be rubblized.

- Regarding subgrade moduli, there was no noted change in subgrade strength before and after rubblization, suggesting little stress dependency occurred.
- Regarding the two types of rubblization equipment, there were no obvious trends or differences in rubblized moduli between the MHB and the RPB. Each type at different times and on various sections produced higher, lower, and similar moduli.

4.5   Other Research Concepts

4.5.1   Retained Modulus Concept

Witczak and Rada in a TRB paper [Witczak et al, 1992] first proposed the "retained modulus" concept. This concept predicts a post-rubblized modulus as a percent of the pre-rubblized PCC modulus. Their initial estimates for highway pavements were in the 8% to 10% range. From the projects already discussed in Section 4.2 and 4.3 whose pre-fractured PCC modulus was known and made available, a data set is compiled in Table 4.11 for "retained modulus" values.

Reference (Site) *Authors: of this report	Location in This Report	Slab Thickness (inches)	Pre - Rubb. Modulus (ksi)	Post Rubb. Modulus (ksi)	Retained Modulus (%)
Anderson (Selfridge)	Table 4.1	16	4370	430	9.8
Anderson (Selfridge)	Table 4.1	21	2730	370	13.5
Anderson (Selfridge)	Table 4.1	13	2050	183	8.9
Galal (INDOT)	in text	8	3882	181	4.7
Galal (INDOT)	in text	10	3692	166	4.5
NAPTF (MRC)	Table 4.6	12	3895	371	9.5
NAPTF (MRS)	Table 4.6	12	4871	289	5.9
*Authors (US 83-TX)	Table 4.7	8	3072	114 (i)	3.7
*Authors (US 83-TX)	Table 4.7	8	3072	199 (f)	6.5
*Authors (LTPP-IL)	Table 4.9	10	5975	146 (i)	2.4
*Authors (LTPP-IL)	Table 4.9	10	5975	233 (f)	3.9
*Authors (LTPP-IL)	Table 4.9	10	5618	100 (i)	1.8
*Authors (LTPP-IL)	Table 4.9	10	5618	184 (f)	3.2
(i) represents an (i)nitial modulus values found immediately after rubblization (f) represents a (f)inal modulus values found several years after rubblization					

Table 4.11, Summary of Retained Modulus Values

Retained modulus values range from 1.8% to 13.5%, with an average of 6.0%. This data is plotted as "retained modulus" versus slab thickness in Figure 4.9. The relationship shows that thicker slabs provide higher retained modulus values. The correlation ($R^2 = 0.69$) is considered fair, but the data is recognized to be very limited for drawing conclusions. Of the 13 data points, three of them represent an initial value (i) where a paired final value (f) determined later on the

same section is also included. Thus, Figure 4.9 represents only 10 different sections. In comparison, Figure 4.8 represents 17 different sections. The retained modulus concept appears promising and should be explored further as more data becomes available. If the pre-fractured PCC modulus is known to the designer, it can be a consideration when selecting a rubblized modulus from the ranges provided earlier (based on slab thickness). When considering the presence of steel in the PCC, the "retained modulus" concept intuitively makes sense since steel should cause an increase in the modulus value both before and after rubblization.

The chart shows data points with a trend line and the equation:

$$\% \text{ Retained} = -2.94 + 0.78 \, H$$
$$R^2 = 0.69$$

Y-axis: % Retained E; X-axis: Slab Thickness (ins)
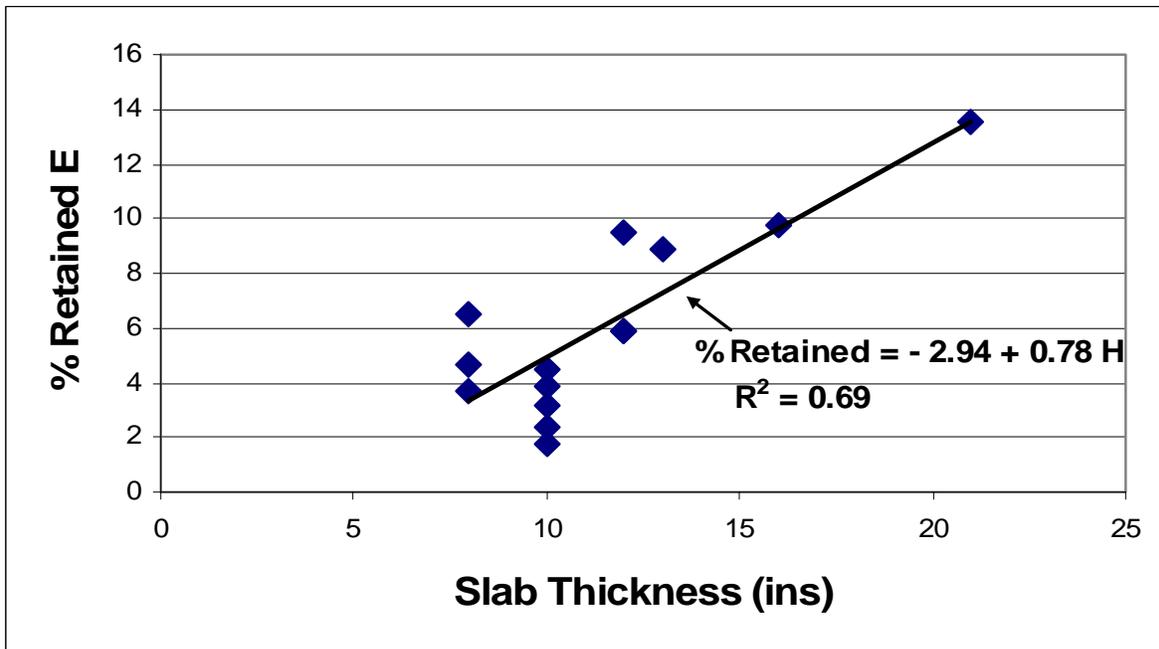
Figure 4.9, "Retained Modulus" Percent versus Slab Thickness

4.5.2   Modeling Rubblized Material as a Two Layer System

As discussed in the Michigan report [Baladi, 2002] and summarized earlier in Section 4.2, the upper part of a rubblized slab often resembles and is characterized as "crushed" material whereas the lower part of the rubblized slab is characterized as "fractured" with substantially less breakage. This observation of two distinct types of behaving material is common with MHB operations and is also observed with RPB jobs on thicker slabs.

The decision on whether to model the rubblized concrete as a composite layer with an average modulus value or two layers with an upper half (low modulus) and lower half (high modulus) has a major impact on the estimated fatigue life of the pavement section. This is demonstrated below by modeling a thick pavement structure using the moduli values recommended by Baladi and presented earlier in Table 4.4. To perform this analysis, a linear elastic analysis program called AIRFIELD was written and used, which also includes the existing LEDFAA design equations for fatigue cracking and rutting.

The sections modeled consist of an HMA overlay (modulus = 400 ksi) of varying thicknesses, all on 17" of rubblized PCC on a subgrade (modulus = 8 ksi). For the rubblized slab, the modulus values reported by Baladi for both a one layer system and a two layer system

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(in Table 4.4) are used. The loading on these sections was arbitrarily selected as a set of dual tires with load per tire of 55,000 lbs, tire pressure of 200 psi and tire spacing of 72 inches. The predicted tensile strains (at the bottom of the HMA) and the fatigue life for these sections are shown in Table 4.12.

	Computed Tensile Strains, in micro-strains (Fatigue Life, in Load Reps x1000)			
Rubblized Layer Modeled As:	4" HMA Overlay	6" HMA Overlay	8" HMA Overlay	10" HMA Overlay
Single Layer, 17-inches (E = 207 ksi)	59 (>100,000)	134 (13,000)	153 (6,700)	149 (7,600)
Two Layers, 8.5-inches each (E = 87 and 503 ksi, respectively)	261 (464)	307 (206)	289 (279)	254 (532)

Table 4.12, Fatigue Modeling with Rubblized Material as Single and Double Layer

The difference in design life is huge between the sections modeling the rubblized material as one-layer versus the two layers. Taking the 6-inch HMA overlay section as an example, the computed fatigue life with a single rubblized layer is 13 million repetitions, compared to only 206,000 repetitions if modeled with two rubblized layers. If the modulus of the top portion of the rubblized layer is halved as Baladi suggests, the reduction in fatigue life is even more drastic.

This is an area where more work is required. Instrumenting future research sections with either strain gauges or multi-depth deflectometers may provide some answers. The degree to which the two layer system proposed by Baladi exists in the field may also be related to the level of rubblization in the field. Repeat passes of the rubblizing equipment, which "powder" the surface, may have implications on the pavement's fatigue life. Modeling the rubblized material as a two-layer system is not recommended at this time.

4.6   CBR Equivalency Factors

If the CBR method of pavement design is used, we recommend the rubblized layer be characterized using equivalency factors. The layer equivalency approach is discussed in detail in Section 8.2 of Appendix A, using only data from the NAPTF sections. Characterizing the rubblized layer as simply a crushed stone base (CBR=100) is very conservative. A more accurate approach would be to characterize the rubblized layer as a stronger material, using the following equivalency factors (EF) based on slab thickness to convert from a rubblized PCC to an "equivalent" crushed aggregate base. These factors consider all the data presented in this chapter.

- Rubblized Layer 6 to 8 inches thick: EF = 1.2
- Rubblized Layer 8 to 14 inches thick: EF = 1.4
- Rubblized Layer > 14 inches thick: EF = 1.6

4.7   Minimum Overlay Thickness

While a mechanistic analysis might suggest that the minimum HMA overlay thickness required to prevent subgrade distortion is minimal in some cases, construction considerations come into play when considering a minimum thickness criterion.   Factors to account for include the type of overlay materials used (HMA, aggregate base, recycled asphalt pavement-RAP, crushed concrete, or combinations thereof) and the ability to successfully place and compact the overlay to meet specified compaction and smoothness requirements.

Other recommendations for minimum overlay thickness have only considered the use of HMA layers directly over the rubblized PCCP. The Asphalt Institute MS-17 states "most agencies have accepted 125 mm (5 in) as a minimum overlay thickness" over rubblized PCCP. FAA's EB-66 currently requires "a minimum 4-inch asphalt surface course" for pavements designed for aircraft with gross weights of less than 30,000 pounds and a minimum of 5 inches for all other cases.

Correspondence and conversation with paving contractors that have worked on highway projects using either rubblization method suggest that at least two lifts of HMA are necessary to meet grading and smoothness requirements, with the first lift being at least 3 inches thick. This agrees with the USAF current recommendation in their ETL 01-09. A minimum of two lifts is necessary because the original concrete pavement can be rather rough, even after rubblization and rolling, and at least two lifts are necessary to meet smoothness requirements.

The need to have at least 3 inches on the first lift of HMA is partly due to factors that inhibit the ability to achieve the desired level of compaction. One factor is that the surface of rubblized material is loose relative to FAA P-209 base material due to the lack of fine particles. This could hinder the HMA compaction process by not allowing sufficient confinement of the mixture. Since applying a tack or prime coat to the rubblized surface is not recommended (see Chapter 6), sufficient lift thickness is necessary to prevent slippage along the interface between the HMA and the rubblized surface during compaction. The USAF recommendations also suggest a minimum lift thickness to minimize the possibility of FOD if aircraft traffic runs temporarily on the surface of the first lift.

Smoothness requirements differ for airfield and highways. However, the need to level the profile and densify the first lift of HMA suggests that the 3-inch minimum requirement for the first lift is appropriate. Assuming this is the case, the minimum recommended overlay thickness should be 5 inches, since the thinnest surface lift recommended is 2 inches using mixture gradations presently available in the FAA's P-401 "Superpave" specification (EB-59a). It is also relevant to note that no airfield project examined in this research project had a final overlay thickness of less than 5 inches.

There have been airfield projects where an unbound base, such as crushed aggregate or crushed concrete, have been placed directly over the rubblized (and rolled) layer prior to placing the HMA. As with an HMA leveling or HMA base course, the ability to place and compact an unbound base to meet in-place criteria are what should govern the minimum layer thickness. Some specifications, such as P-208 (Aggregate Base Course), require a lift thickness between 3 and 6 inches, while P-209 (Crushed Aggregate Base Course) sets a <u>maximum</u> compacted lift thickness requirement of 6 inches. When any unbound base is placed directly on top rubblized/rolled material, this research team recommends a minimum unbound layer thickness of

4 inches. On-site mixing of this unbound layer directly on the rubblized/rolled surface is discouraged because problems could occur with disturbing the larger pieces near the surface with the mixing or grading equipment. Placing this unbound layer with an HMA paving machine is recommended to achieve greater smoothness and to avoid unnecessary disturbance. Placing a prime coat on this unbound base is recommended prior to placing HMA, even though priming is not recommended directly on rubblized material. The difference is the lack of fines and variable surface texture that typically exist on a rubblized surface, making the functionality of a prime coat very difficult.

When an unbound base is placed over a rubblized layer, the minimum HMA thickness requirement that exists for the unbound material should apply. For general aviation airfields, this criterion is currently two 1½-inch lifts for a total of 3 inches for airfields receiving aircraft traffic under 30,000 lbs gross weight.

If placing HMA directly over rubblized material, the minimum HMA total thickness is 5 inches, placed in two lifts with the first lift being at least 3 inches. Care must be taken when selecting HMA lift thicknesses and mix designations (by particle size) to ensure sufficient lift thickness for adequate particle reorientation during compaction. Otherwise, sufficient density will not be achieved and/or the aggregate in the HMA will be crushed during the rolling operation. In either case, the minimum overlay thickness requirement should be the absolute minimum used on the project, which is an important consideration where significant leveling and variable overlay thicknesses are required. Designers should consider this when developing their final grading plans for rubblized airfield pavements.

4.8   Thickness Design Sensitivity Analysis

To determine how the range of rubblized layer moduli suggested in Section 4.4 will effect the HMA overlay thickness design requirements, a sensitivity analysis was conducted for a wide range of design conditions. The analysis was conducted using LEDFAA using the following variables:

- Three subgrade strengths: 6,000 psi, 12,000 psi, and 18,000 psi
- Three modulus values for the rubblized PCC layer: an extremely high value and a low-moderate value given the ranges suggested in Section 4.4, along with a value assigned by LEDFAA when treating the rubblized layer as P-209 base material. LEDFAA automatically computes a modulus of the P-209 material as a function of layer thickness and the modulus of underlying layers [Barker, 1975]
- Three slab thicknesses typical of three categories of service: 16" for heavy traffic (major hub), 12" for medium traffic (commercial), and 7" for light traffic (general aviation - GA)
- A typical aircraft traffic mix representative of each of the three airport service categories

The traffic mix data are contained in Table 4.13. The results of the overlay computations are summarized on Table 4.14, with the results depicted graphically on Figures 4.9, 4.10, and 4.11 for the heavy (16" PCC), medium (12" PCC), and light (7" PCC) pavements, respectively. Design HMA overlay thicknesses that are less than 5-inches, which is the minimum HMA overlay thickness recommended in Section 4.7, should default to this minimum.

No.	Name	Gross Wt. lbs	Annual Departures	% Annual Growth
A. Heavy Mix, IAD				
1	C-130	155,000	1,137	0.00
2	B-707	335,000	1,870	0.00
3	Dual Whl-50	45,000	16,711	0.00
4	Dual Whl-75	75,000	81,629	0.00
5	A320	160,000	30,138	0.00
6	B-717	121,000	2,895	0.00
7	B-727	210,000	910	0.00
8	B-737-800	175,000	6,498	0.00
9	B-757	250,000	14,899	0.00
10	DC-9-50	122,000	3,136	0.00
11	DC-8	358,000	191	0.00
12	B-767-200	335,000	1,333	0.00
13	B-767-300ER	409,000	1,453	0.00
14	B-777-300ER	750,000	7,892	0.00
15	B-747-400	873,000	1,426	0.00
16	MD-11	621,000	704	0.00
17	MD-11 Belly	621,000	704	0.00
18	A300-B4	331,000	763	0.00
19	A330	459,000	1,280	0.00
20	A340-200/300	567,000	1,580	0.00
21	340-200/300 Belly	567,000	1,580	0.00
22	A380-800F	1,305,125	734	0.00
B. Medium Mix, CHO				
1	B-737-300	150,000	12,775	0.00
2	B-737-400	140,000	3,650	0.00
3	B-737-700	160,000	183	0.00
4	Fokker F100	100,000	1,095	0.00
5	A320	150,000	2,372	0.00
C. Light (General Aviation) Mix, Typical				
1	Dual Whl-50	36,314	100	0.00
2	Dual Whl-75	68,540	52	0.00
3	Dual Whl-75	75,000	12	0.00
4	Dual Whl-50	48,200	100	0.00
5	Sngl Whl-30	18,000	300	0.00

Table 4.13, Traffic Mix Data for Sensitivity Analysis

Calculated HMA Overlay Thickness Required (inches)	Thickness of Rubblized Layer (inches)	Modulus of Rubblized Layer (psi)	Modulus of Subgrade (psi)
	(Heavy Traffic Mix)		
32.0	16.0	As P-209	6,000
17.0	16.0	As P-209	12,000
8.0	16.0	As P-209	18,000
21.5	16.0	300,000	6,000
9.5	16.0	300,000	12,000
2.5	16.0	300,000	18,000
15.5	16.0	700,000	6,000
4.5	16.0	700,000	12,000
2.0	16.0	700,000	18,000
	(Medium Traffic Mix)		
15.5	12.0	As P-209	6,000
9.0	12.0	As P-209	12,000
5.5	12.0	As P-209	18,000
11.0	12.0	150,000	6,000
6.5	12.0	150,000	12,000
3.0	12.0	150,000	18,000
8.0	12.0	300,000	6,000
4.0	12.0	300,000	12,000
2.0	12.0	300,000	18,000
	(Light Traffic Mix)		
8.5	7.0	As P-209	6,000
5.5	7.0	As P-209	12,000
3.5	7.0	As P-209	18,000
6.0	7.0	100,000	6,000
3.5	7.0	100,000	12,000
2.0	7.0	100,000	18,000
5.0	7.0	150,000	6,000
2.5	7.0	150,000	12,000
2.0	7.0	150,000	18,000
Notes:			
1. Analysis done using LEDFAA pavement design procedures			
2. For the 16" PCC runs, a 5" thick P-401 layer was assumed to be over a P-403 layer.			
3. For the 12" and 7" PCC runs, a 4" thick P-401 layer was assumed to be over a P-403 mix.			
4. For the 16" and 12" PCC runs, a 6" thick aggregate base (P-209) was assumed to be under the rubblized PCC layer. For the 7" PCC runs, no P-209 base was assumed.			
5. A modulus of 200 ksi was assumed for both the P-401 and P-403 overlay layers.			
6. Calculated HMA overlay thicknesses rounded to nearest 1/2 inch.			
7. No minimum HMA overlay thicknesses applied.			

Table 4.14, Results of Sensitivity Analysis for HMA Design Overlay Thicknesses
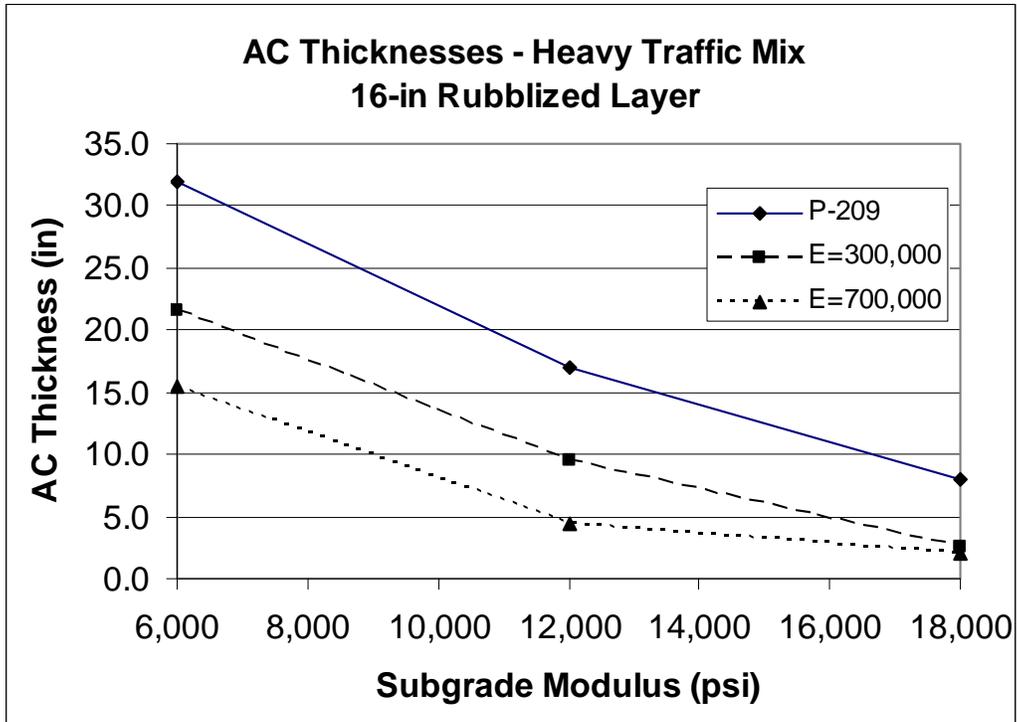
Figure 4.10, HMA (AC) Design Overlay Thicknesses for Heavy Traffic Mix
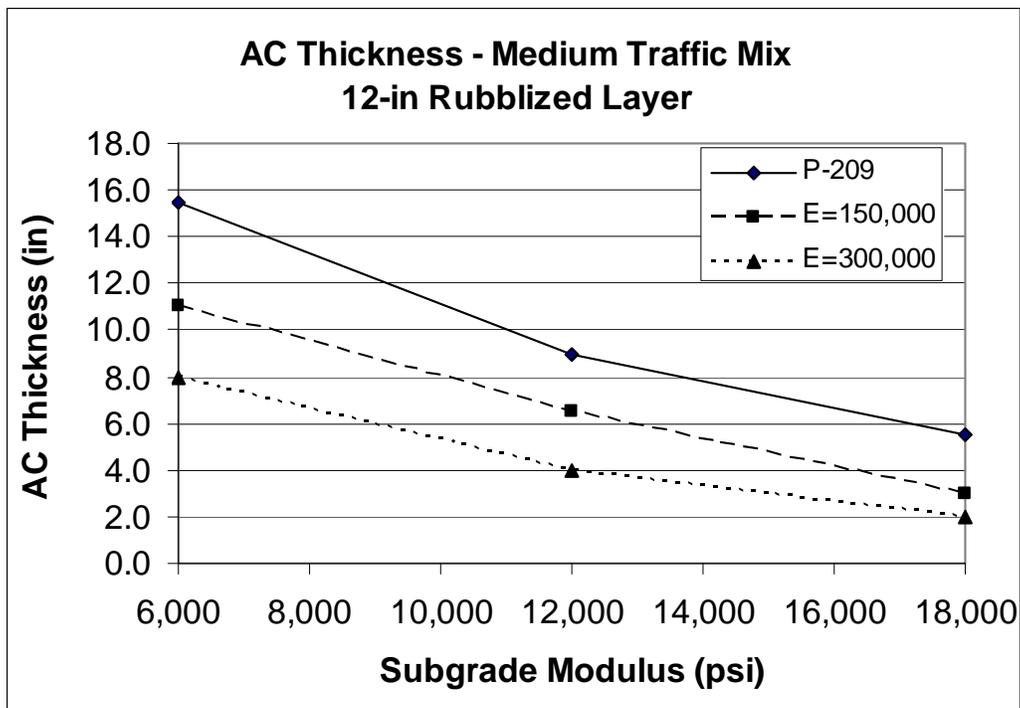
Figure 4.11, HMA (AC) Design Overlay Thicknesses for Medium Traffic Mix
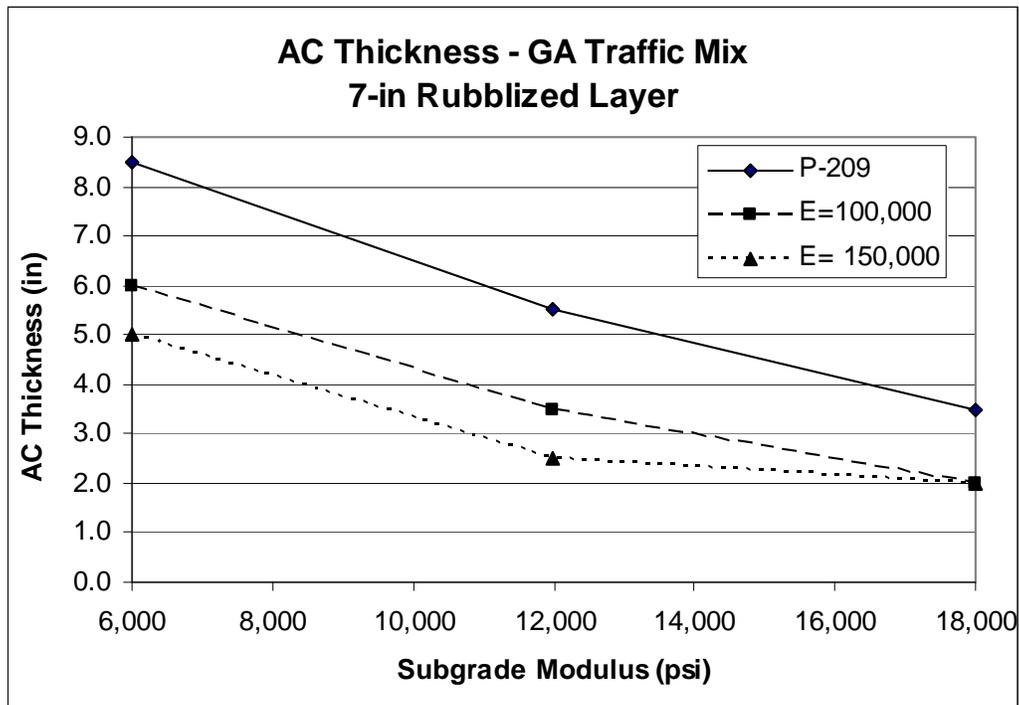
Figure 4.12, HMA (AC) Design Overlay Thicknesses for Light Traffic (GA) Mix

As can be seen from Figures 4.10, 4.11 and 4.12, the HMA overlay design thicknesses are generally reduced between 50 to 80 percent by using our recommended range of modulus values for the rubblized layer (from Section 4.4) versus characterizing as a P-209 base within LEDFAA. The minimum overlay thickness requirements on rubblized material often nullify much of this reduction, especially for GA airfields. It is also interesting to note how sensitive the HMA overlay thickness requirement is to subgrade modulus.

CHAPTER 5 - QUALITY ASSURANCE PRACTICES AND FUTURE CONCEPTS

This chapter begins with an introduction on specification types and quality assurance methods used for assuring effective rubblization. The next three sections discuss and provide recommendations on practice and criteria regarding three major components of rubblization quality assurance: 1) test strips, 2) test pits and 3) particle size requirements. The last section explores future concepts of utilizing in-situ deflection testing to control rubblization quality.

5.1   Introduction

Transportation Research Circular E-C074 [TRB, 2005] provides definitions for terminology associated with highway construction specifications. According to TRB, a "performance-based" specification uses tests that measure fundamental engineering properties such as modulus of elasticity for quality control and assurance, while a "performance-related" specification relies on tests that are indicators of properties directly related to pavement performance. A "method" specification describes equipment and/or construction practices employed, basing acceptance on compliance with those practices. The rubblization specifications reviewed in Section 2.5 of this report are best classified as "method" type specifications since they emphasize processes and equipment. They also include "performance-related" features by setting particle size requirements and requiring any steel to be debonded from the concrete.

The intent of rubblization is to eliminate "slab action", converting the slabs into a stable, unbound base layer. Limiting the particle sizes and requiring that steel (temperature, reinforcing and dowel bars) be debonded are the prevailing specification criteria for assuring no slab action. Verifying these criteria requires the excavation of a test pit. It is important that the specification either references a procedure or provides some guidance describing how these requirements may be ascertained. The in-place properties of the rubblized layer can affect the ability to successfully place and compact the overlay layers as well as overall pavement performance.

There does not appear to be any specification that includes an in-place density requirement for the finished rubblized PCCP layer. The reason is that there is no way rubblized material can be sampled and tested for developing a moisture-density relationship used to set a minimum in-place dry density requirement. Removing a sample of the interlocked rubblized material would disturb it in such a way that the measured properties would not be representative of in-place characteristics. Additionally, the extent of fracture can differ significantly depending on the depth within the rubblized layer. It should also be noted that the rolling required as part of the rubblization process is not intended to further densify the rubblized surface, but rather to seat and provide a uniform and stable surface prior to an overlay.

While many assume that both types of rubblization equipment are somewhat "self-regulating," and many specifications rely on this assumption, we recommend utilizing a test strip and test pit. The assumption is that if the slab is not completely fractured, the breaking head on the RPB or the hammers on the MHB will reverberate in such a manner that it will be readily apparent the breakage energy is not being absorbed into fractured PCCP, thus not breaking the slab. While the literature mentions this "self-regulating" feature often and we believe this to be

true to some extent, there have been projects where the rubblization results were deemed unacceptable after digging the test pit, even though the rubblization operation was not halted beforehand. As pointed out in earlier chapters, these occasional problems with achieving effective rubblization are not exclusive to either the MHB or the RPB. Some type of real verification is necessary to ensure adequate full depth fracture within the PCCP and debonding of the steel.

Identifying weak rubblized sections that may fail to support construction traffic is another desirable function of the quality control process. Chapter 3 discussed the potential problem of rubblizing thin PCCP (eight inches or less) with little or no base support. There have been instances where rubblized material and support layers were not stable enough to support heavy construction equipment even though everything appeared to be normal. It is in the best interest of all parties to be able to identify these marginal support conditions as early as possible. Proof rolling as part of a quality control process is one such way. This was addressed in Chapter 3.

Aside from digging test pits and examining particle sizes, another means of evaluating the extent of rubblization and verifying the stability of the rubblized section is to measure in-place deflections on the rubblized/rolled surface. From this, it might be possible to:

- judge whether the rubblized layer is bending or distributing loads through shear
- determine if the composite section is stable enough for construction traffic
- check conformance with layer modulus values assumed during structural design

Prospective approaches for using this "deflection measurement" concept as a means of quality control or acceptance requirements are discussed at the end of this chapter in Section 5.5.

5.2   Test Strips

The purpose of a test strip and test pit is for the contractor to demonstrate effective rubblizing and rolling practices for conditions representative of the entire pavement section. A test pit is excavated from within the test strip area to verify that the slab was fractured full depth to the conditions called for in the specification, namely with regard to particle size and steel debonding. Besides providing a rubblized section from which a test pit can be excavated, the test strip provides the opportunity to evaluate and optimize the operation of equipment used to rubblize and roll the PCCP. This is a chance for the rubblization contractor to ensure they have the proper equipment and to "fine-tune" the operation to achieve the required result, along with getting an idea of production rate. The breaking equipment can be adjusted in terms of drop height, load amplitude, frequency, speed, tires, etc. Rollers can be adjusted in terms of vibratory or static mode, number of passes, speed, etc. The test strip should be proof rolled to assess its ability to support construction traffic during subsequent paving operations, especially if HMA (versus aggregate base) is to be placed directly over the rubblized and rolled layer.

The two airfield rubblization specifications, FAA's P-215 (part of EB-66) and USAF's ETL 01-09, have different dimension requirements for the test strip. The FAA requires 150 ft long by 12 ft wide while the USAF requires 500 ft by 12 ft. RMI suggests a length of 300 ft in their guide specification. Neither specification is explicit as to whether a separate test strip is required for each pavement "feature." The term "feature" in this chapter will mean any unique pavement cross-section profile in terms of PCC or support layer types/strengths/thicknesses.

Considering the variety of pavement features that can exist on an airfield, we recommend that a new test strip (and test pit) be performed when different conditions of slab thickness,

reinforcing steel, support layers or drainage are encountered. How features are differentiated may vary from project to project, but could include subtle differences. This point is illustrated by reviewing a 2006 rubblization project at Kegelman Auxiliary Field in Enid, OK. The PCCP runway had only 5 to 6-inch thick slabs with 0 to 8-inches of marginal sandy subbase. While the single test strip, located on the edge of the runway, revealed no problems with rubblizing these marginal conditions, unstable areas and punch-thrus became more and more frequent as the process moved toward the centerline. Personnel involved with this project attributed this to a subgrade moisture content that seemed to be greater further from the runway edge. Significant cost and time overruns occurred by having to replace the ineffective rubblized PCCP and three feet of subgrade with crushed concrete for about 30% of the project area. A second test strip, closer to the centerline at the beginning of the project, could have been very beneficial in order to anticipate and plan for the large amount of full depth repairs.

For thicker slabs, the test strip is often located on the furthest outside slab of a feature because a free edge greatly facilitates the excavation of the test pit where the bucket can get underneath the larger PCC pieces. This is less of an issue for thinner slabs where a backhoe bucket can more easily penetrate the rubblized layer. While the pavement edge is convenient for excavation purposes, it may not be the most representative in terms of support and drainage conditions.

Summary of Recommendations on Test Strips:
- Allows observation of rubblizing and rolling operations to assure suitable surface texture
- Provides rubblized area for excavation of test pit
- Should be a minimum 300 ft in length and one slab dimension wide
- One test strip for each unique pavement feature (cross section profile) designated in plans
- Apply proof rolling to observe stability of rubblized layer
- Select test strip location representative of the entire feature

5.3   Test Pits

Current specifications generally rely on observations from a very small sampling of the project (4 ft by 4 ft test pit within a test strip) to determine compliance against a maximum or nominal maximum particle size requirement. These do not refer to defined test procedures for measuring the particle sizes objectively. This is because the excavation and examination of test pits is an arduous process that requires significant work during, as well as afterwards, to repair the sampled area. While the rubblization process is somewhat self-regulating for the vast majority of operations, test pits are still highly recommended.

Michigan DOT's specification places much greater emphasis on the evaluation of test pits for acceptance, requiring a test pit to be excavated at least every 1500 ft and detailing how it is to be excavated to assure the desired maximum particle size is met and that the steel is completely debonded. This was discussed in Chapter 4. These test pit requirements are much more rigorous compared to those of the Arkansas HTD and Louisiana DOTD. Even so, the incidence of reported pavement performance problems on rubblized projects appears to be greater in Michigan than in Arkansas or Louisiana, although those documented problems are more construction related than from rubblization procedures.

An issue regarding test pit acceptance criteria is whether the specification and enforcement should require "complete" steel debonding or some lesser degree. On one extreme is the strict Michigan enforcement of complete debonding, while most other states accept a lesser degree. Depending on the type of steel present and the conditions, 100% debonding is usually difficult to achieve. Repeated passes of the rubblizing equipment may help achieve additional debonding, but may also produce a less desirable effect of reduced in-place modulus. This was discussed in Section 4.3.2 of this report. It is always important to remember that the objective of rubblization is to eliminate slab action while retaining a strong support layer for an overlay. We believe it is appropriate to use the term "substantially debonded" in a rubblization specification and let engineering judgment prevail when examining test pit conditions to determine if slab action is eliminated. A minimal amount of PCC bonded to the steel should not cause reflective cracking as long as slab action is eliminated. The finer rubblized material above the steel may also serve as a crack relief "interlayer." Figures 5.1 and 5.2 below show a steel rebar mat that we consider "substantially" debonded from the PCC. On the other extreme, Figures 2.2 and 2.3 from Chapter 2 show test pits in Michigan where the temperature steel remained substantially bonded to the concrete and could potentially cause a reflective cracking problem. In the case of dowels bars, the contractor should be allowed to sawcut along the joints prior to rubblization to sever the dowels if these cannot be "substantially" debonding. Figure 4.4 illustrates such a condition in one of the NAPTF test pits where sawcutting along the joint would have been recommended.

Figure 5.1, Substantial Debonding of the Steel Rebar

Figure 5.2, Test Pit with Rebar Mat Substantially Debonded

Both current airfield specifications require a 4 ft by 4 ft test pit to be "excavated in the middle of the test strip." It is not clear in either case if this means in the middle of the width or middle of the length. There is currently no certainty whether the conditions at a slab edge (where there can be dowels or greater thickness) or slab interior are properly evaluated. To address these issues and potential concerns with properly patching the area, we suggest the test pit be across the entire slab width of the test strip, be at least 6 ft long (longitudinal to the direction of rubblization equipment), and include both a transverse and longitudinal PCCP joint. The larger test pit will allow a more thorough evaluation of the effectiveness of the fracturing operation for slab edge and slab interior conditions. The larger test pit will also allow heavier compaction equipment be used to facilitate better compaction of the patch material. All the layers should be compacted as they are placed, starting with the subgrade surface.

Both the FAA and USAF specifications require the use of "full-depth bituminous concrete" to patch unstable areas, but require the use of "coarse aggregate material" for patching the test pit excavation. Requiring full-depth HMA should also be considered to fill test pits to avoid creating a soft spot, especially when an HMA layer is to be placed immediately after rubblizing and rolling. In this case, the materials and equipment necessary to place and compact HMA will likely be onsite and could be employed. If an unbound aggregate base is to be placed

directly over the rubblized PCCP, it will likely be more feasible to use the same material for patching the test pit.

Current specifications do not state whether the test pits should be excavated before or after the rolling process. Future specifications should say that the test pits are excavated after the entire rolling process, including proof rolling. One reason is to assure that the entire fracturing process is evaluated, which includes the Z-grid roller for the MHB. Another reason is to make sure the rubblized layer has not been contaminated with fines from an underlying layer and there has been no shear distortion of underlying layers. While unlikely, this could be caused by excessive vibratory rolling on relatively thin (<8 in) rubblized slabs with no base or subbase (see Section 3.3.8 *Turn Vibrators Off on Rollers*).

Summary of Recommendations on Test Pits:
- Excavate test pit after fracturing, rolling and proof rolling has occurred within test strip
- Desired dimensions are the full slab width of the test strip and at least 6 ft long
- Locate the test pit so that it includes a transverse joint and a longitudinal joint
- Avoid locations that are near the wheel-path of normal aircraft traffic
- Examine test pit rubble to determine if specification criteria have been met, including:
    - fracture through full depth of PCC slab
    - steel "substantially" debonded from concrete (dowels may be sawed at the joint if can't achieve "substantial" debonding)
    - particle size requirements are met (see next section)
    - no contamination of subgrade fines into rubblized layer
    - no shear distortion of the subgrade
- If the test pit is located along a free edge (such as the outside edge of a feature), the sizes of the broken PCC pieces may be larger and "out of spec" at the outside edge due to the lack of lateral support during rubblization. The outside edge for this purpose is defined as two to three times the slab thickness. This "out of spec" provision should be allowed. Locating the test pit along a free edge is more typical of thicker slabs.
- Remove and dispose of material from test pit
- Repair test pits by carefully compacting patch material (high quality crushed aggregate or HMA) in lifts per requirements specific to the material used. Using HMA (versus aggregate) as a patch material should help avoid creating a "soft spot" from the patch.

5.4   Particle Size Requirements

Factors affecting particle size include PCCP factors (thickness, material properties, steel), support conditions, equipment type and operation, and location within the slab. The FAA specification (P-215) sets different particle size requirements depending on presence of reinforcing steel, depth within the fractured slab, and location relative to a pavement edge. The P-215 requires smaller particle sizes in the upper half of the slab (75% of material < 3 inches) than in the lower half (75% of material < 12 inches), and permits pieces up to 15 inches in any dimension under reinforcing steel or within 15 inches of the pavement edge due to lack of support.

The discussion of this issue in FAA's EB-66 and RMI's guide specification suggest tying particle size requirements to slab thickness. We agree with expressing the criterion in this

manner, especially for an airfield specification since slab thicknesses vary greatly for airfields relative to highways. The criteria this research team recommends in the bottom half of the slab (or below steel) is that virtually no particles should be greater than two times the slab thickness, up to a maximum of 24-inches, in any dimension. While this criterion allows particles larger than many highway specifications, especially for thicker PCCP, there have been no indications that pieces this large cause reflective cracking (see Grand Forks AFB runway project in Section 2.4). One must remember that the whole purpose of the criteria is to prevent reflective cracking and engineering judgment should be used when enforcing these criteria. That is the purpose for the word "virtually," to allow for the very rare occasion of an oversized piece. This rare occasion may be more likely to occur on thicker slabs. These oversized pieces should not be removed, but rather left in place.

        While most rubblization specifications reviewed describe a maximum or nominal maximum particle size requirement, none referred to a test procedure other than "visual observation." The Federal Interagency Sedimentation Project (FISP) developed a standard for measuring particle sizes of cobbles and gravel materials in streambeds that may be useful when examining rubble in a test pit [Potyondy and Bunte, 2002]. This standard uses templates such as shown in Figure 5.3. These templates are available from FISP at the website shown in the reference or could be fabricated. An alternative to the FISP template is ASTM D 5519, "Particulate Size Analysis of Natural and Man-Made Riprap Materials." While it is probably not feasible to mandate such a standard, especially on thicker slabs, this does provide a more objective basis for making a pass/fail judgment regarding particle size.
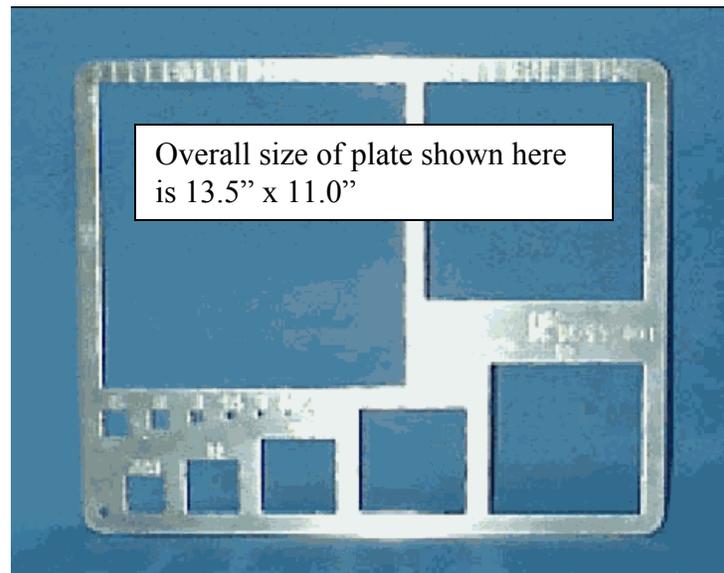
Figure 5.3, US-SAH-97 Templates (reference: Potyondy and Bunte)

Summary of Recommendations on Particle Size Acceptance Criteria:
- Upper half of slab: No particles > 6 inches in any dimension, and at least 75% of material (by weight) < 3 inches in any dimension
- Bottom half of slab or below steel: Virtually no particles > than 2X the slab thickness, up to 24 inches, in any dimension

5.5 Future Concepts for Quality Control/Acceptance

While test strips and test pits are currently the acceptable and appropriate practice for assuring the slabs are adequately shattered, there are some obvious limitations. The time and effort required to excavate and patch a test pit can pose serious practical problems on some projects. Another problem with relying solely on a test pit is that this discreet point may not necessarily represent conditions throughout the section, particularly for very thin or very thick slabs. Changes in subsurface drainage, foundation stiffness or concrete material characteristics can affect the effectiveness of rubblizing. Ideally, acceptance requirements would include a means of global (surface-wide) testing that identifies: 1) whether slab action has been eliminated to prevent the development of reflective cracking and 2) any weak areas that need repair before paving operations proceed. This is particularly important for projects on marginally stable PCCP sections, and that use the MHB without proof rolling.

The new AASHTO mechanistic-empirical pavement design guide suggests for the highest hierarchical level of overlay thickness design that modulus values be determined from deflection measurements. The movement toward mechanistic-empirical structural design procedures for highways and airfields suggests that future specifications consider how to assure that the engineering properties assumed in design are met in the field.

The predominant equipment used to obtain design modulus values is a falling weight deflectometer (FWD). A heavy weight deflectometer (HWD) operates on the same principles but is capable of applying loads more representative of wheels from heavy aircraft. For an FWD or HWD to be used for quality acceptance, a key would be to identify an acceptable range of deflections (i.e. sensor 1) or composite stiffness measurements (i.e. Impulse Stiffness Modulus or ISM). Initial implementation of this technology could still utilize a test strip and test pit, but also run the FWD on the approved test strip to obtain a reference set of measurements (sensor 1 deflection or ISM). The FWD could then be run over the entire section to determine if the response measurements fall within an acceptable range established relative to the reference set.

An illustration from real data on how an FWD can be used for QA/QC is illustrated in Figure 5.4. The Michigan DOT tested I-75 after placement of the first lift of HMA. This data suggests the section around Station 3 may not have been effectively rubblized with a deflection less than 4 mils. The very high deflections at Stations 10 and Stations 13-16 are in an area where extensive full depth repairs were made after rubblization and the rubblized PCC was removed. After Station 20, the deflections are relatively constant.
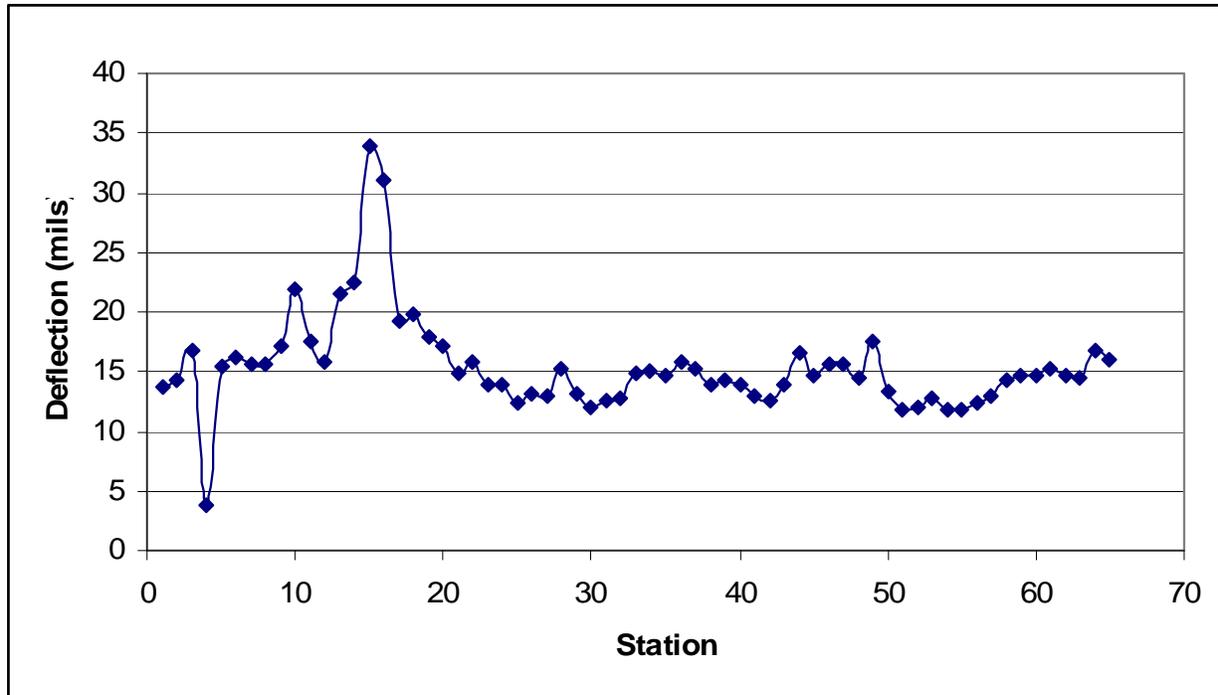
Figure 5.4, FWD maximum deflection data from I-75 in Michigan

The FWD should be considered as a quality assurance tool on future rubblization projects. In the simplest manner, it could be used in the early days of a project to identify potential "unrubblized" areas, identify localized weak spots, and ensure the rubblized concrete is providing uniform support. A more analytical approach could utilize a basin parameter such as area, basin slope or surface curvature index to verify that the fractured PCCP no longer functions as a slab. An even more futuristic approach could consider establishing a range of target backcalculated moduli that the rubblized layer would need to meet. While using basin parameters or backcalculated modulus values obtained from complete deflection basins has benefits, this approach is not currently practical when performing the tests directly over rubblized and rolled PCCP. For sensors located away from the loading plate, the coarse and uneven surface of rubblized PCCP makes seating the geophones very difficult and results in errors.

Procedures that measure deflection under load in a continuous manner may provide the best measurement for future quality control and acceptance of rubblization. NCHRP 10-65, *Nondestructive Testing Technology for Quality Control and Acceptance of Flexible Pavement Construction,* is a project currently in progress to evaluate a wide variety of technologies that could be used to characterize flexible pavement layers and begin the process of developing performance-based acceptance criteria. The procedures recommended from this study that apply to unbound materials may also be applicable to rubblized PCCP, although the acceptance requirements would need to be established separately. Using some form of deflection measurement to estimate a composite or layer modulus would also require that the procedure be calibrated to more familiar test methods.

In France, a device called a "Portancemètre" has been developed that generates vertical movement using an eccentric mass, measuring the response with accelerometers similar to how a

Dynaflect operates. This equipment operates as it is being towed across the surface, allowing a nearly continuous reading of deflection or modulus with distance. The website for the "Laboratoire Central des Ponts et Chaussees" has additional information about the Portancemètre Continuous Capacity Measurement device [http://www.lcpc.fr].

In several European countries, "Intelligent" compaction equipment or other test procedures that measure deflections are included in the acceptance testing for base, subbase and subgrade materials. Intelligent compaction technology includes a drum of a vibratory roller that is instrumented to allow the estimation of a composite stiffness of the underlying materials. When combined with monitoring the location using global positioning system equipment, it is possible to obtain a recording of the composite stiffness across the entire surface that was rolled. Intelligent compaction is receiving more interest in the United States and may be an option in the future to consider for quality control or assurance techniques. While this technology has thus far been mostly been used on soils and aggregate base materials, it might be also be feasible to use on rubblized PCCP to identify either potentially weak spots or slabs that were not entirely fractured. The website for the Center for Intelligent Geo Construction at the Colorado School of Mines includes a list of references and roller manufacturers [http://egweb.mines.edu]. The Minnesota DOT also maintains a website with many references for continuous compaction control and other in-situ tests used to measure or estimate stiffness [http://www.mrr.dot.state.mn].

CHAPTER 6 – OTHER ITEMS

This chapter offers additional guidance on 11 rubblization items. There is considerable referencing to earlier sections of this report that also discuss these items, but not from the same perspective as this chapter. For example, the specification comparison that was part of the literature review (Section 2.5) will often be referenced in this chapter since it summarizes and compares how many of these items are handled in current rubblization specifications. This chapter builds on that information to provide additional commentary, guidance, or suggested changes. There are other sections of this report that will also be heavily referenced.

6.1 Drainage Systems

Section 2.5.4 provides a review of current rubblization specifications requirements for drainage systems. Most highway pavements that have been rubblized have included the establishment or improvement of a current longitudinal drainage system. Exceptions have mostly been for older "thickened edge" pavement slabs, or cases where the PCCP was placed directly on the subgrade and there was no base layer to drain.

In general, we recommend the installation of a longitudinal edge drain system for airfield rubblization projects. The system should be functioning at least two weeks prior to the start of rubblization. Other sections of this report, namely Chapter 2 and 3, have discussed the importance of proper drainage in the layers under the PCCP to allow for proper rubblization. The drainage system will also help provide good long-term pavement performance. A typical edge drain system schematic is shown in Figure 6.1. It is important that the edge drain be placed right next to where the rubblization layer will be constructed and extends to the top of the rubblized layer, referred to as the permeable zone in Figure 6.1, because this top material will be the most permeable. The system is intended to drain free water not only from the layers below the rubblized layer, but also any water that may enter the rubblized layer itself. Provisions for cleanouts should be provided in the design in order to allow for proper maintenance later.
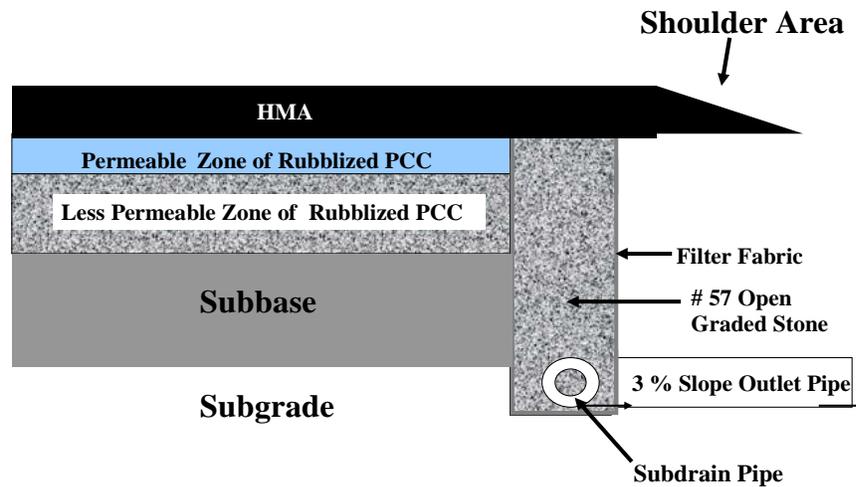
Figure 6.1, Typical Edge Drain System

Lateral drains may be considered at sags in vertical curves, interfacing pavements, etc. The Grand Forks AFB RW project (Section 2.4) had an existing centerline drain that the designer decided to rebuild. Some airfield apron projects have used cross drains (i.e. Ft Wayne ANG Base). Subsurface drainage requirements should be evaluated on a case-by-case basis.

6.2   Underground Utilities and Structures

The owner should identify items that need to be protected prior to rubblization and address them during project design. The contractor is then responsible to operate equipment in a manner that will not damage those items. Full depth isolation saw cuts are typically used as a means of protecting existing utilities, electrical fixtures, drainage inlets, adjacent pavements or other structures. The saw cutting protects the features from any impact or vibration damage that could result from the rubblization process.

For new trenching or utilities, saw cut the PCCP around the edges of the utility prior to rubblization. Rubblize up to the saw cut, and then carefully lift and remove the unrubblized PCC. The pipe or other utility can be installed before repairing with HMA or P-209. For existing utilities, either two parallel diamond blade saw cuts or a wheel saw relief trench should isolate the utility (Figure 6.2). For adjacent PCCP that is to remain intact, a full-depth saw cut is required along an existing joint and any load transfer devices or tie-bars need to be severed.
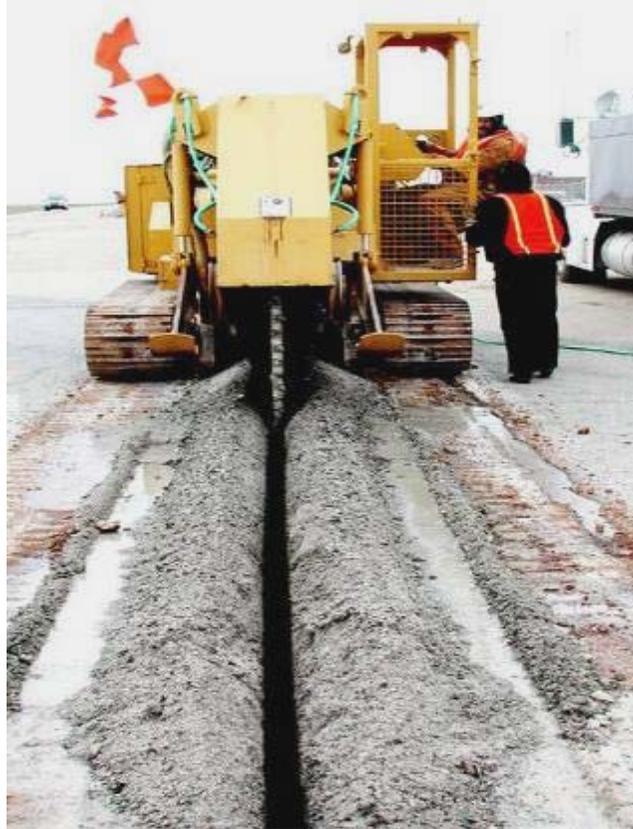
Figure 6.2, Isolation Cut and Relief Trench by Wheel Saw

6.3   Rubblization Equipment

As mentioned in the specification comparison of Section 2.5.3, the descriptions provided in the FAA's P-215 and the Air Force's ETL 01-09 for the types of rubblization and rolling equipment are practically identical. Any type of breaking equipment must be "self-contained, self-propelled" to rule out the use of impact roller devices (i.e. *Impactor 2000),* wrecking balls, etc. These impact roller devices were developed for demolition and removal, and should never be used for rubblization.

Resonant Machines Inc's (RMI) guide specification allows for variation in the frequency and for greater amplitude compared to the current airfield specifications (P-215 and 01-09). These differences exist because the RMI specification is more recent than P-215 or 01-09 and reflects recent modifications to existing RPBs and the most recent addition to their RPB fleet, the PB-500. It would be appropriate to use RMI's description in any revision to P-215.

The descriptions used for the MHB in both the P-215 and 01-09 specifications appear to be the most current. Antigo Construction Inc's (Antigo) website (www.antigoconstruction.com) describes the rubblization work performed at Selfridge ANG base, where a guillotine-type of drop hammer was used to pre-fracture the thick concrete slabs before rubblization. The owner of Antigo states that the guillotine hammer is generally necessary on PCCP greater than about 13 inches thick. It has apparently not been necessary to describe a guillotine hammer or its use

within previous rubblization specifications because the particle size requirements have been enough to drive the decision whether to pre-fracture the slabs with the guillotine hammer.

In terms of guidelines for specifying one type of equipment or another, there are examples of success and failure using both types of equipment over a range of conditions.   For moderate PCCP thicknesses (8-13 inches), either type of equipment can normally fracture the concrete to meet the particle size requirements. Many of the problems with either type of equipment have occurred on very thin PCCP where extremely soft support conditions existed. Chapter 3 covers this issue in detail. Each type of equipment (MHB and RPB) has replaced the other on projects for various reasons ranging from poor fracturing to slow production. Since there is no consistent pattern as to when one type of equipment performs better than the other, it is preferable to focus on the result rather than the process in the specification. This is covered extensively in Chapter 5 on quality assurance. It also appears there are cost advantages to allowing both types of equipment, which is addressed later in this chapter.

6.4   Rolling Equipment and Process

For lack of a procedure to gauge the "tightness" of the rubblized and rolled surface, rubblizing specifications typically provide descriptions of equipment and procedures used for rolling the fractured surface. Existing specifications require different types of rolling/seating equipment depending on the type of breaking equipment used. The rolling equipment required when using the RPB describes similar rollers as used for compacting HMA, while rubber-tired rollers and vibratory steel drum rollers are required when using the MHB.

P-215 requires the use of "smooth double steel drum vibratory roller" as "seating equipment" when the RPB is used. In reality, any "seating" that occurs takes place as the RPB works back and forth over the pavement. The roller finishes the surface so that the HMA paving operation can take place with minimal difficulty. Consequently, it would be best to refer to "rolling" equipment and procedures rather than "seating" equipment.

As for the roller requirement, it would be best to describe the equipment as a "steel drum vibratory roller," and require that the drum or drums operate in vibratory mode at a frequency of more than 2500 vibrations per minute (41.7 Hz). P-215 currently requires a double drum vibratory roller in conjunction with the RPB, and in our opinion a single drum vibratory roller is sufficient. Rollers should operate at a forward speed to assure that at least 10 impacts per foot are applied to the rubblized surface. These requirements suggest the same rollers and operations as used for compacting HMA.

As noted in the review of existing specifications in Section 2.5.3, most specifications including P-215 include requirements for the minimum number of passes using the prescribed roller types. This appears to have worked successfully and should be carried forward into any revised specification. However, no specification requires sprinkling or watering the surface before rolling. Even though the rolling is not intended to achieve a target density, moisture would undoubtedly help reorient superficial particles, tightening the surface texture in preparation for paving.

Prior to accepting the rubblized and rolled PCCP, it may be beneficial to proof roll the surface to assure that there are no soft spots that might affect the HMA paving and compaction operation or the long-term performance of the pavement. This is particularly important when the MHB is used on relatively thin pavement structures (< 9 inches) slabs, since the equipment is

operated on the intact (non-rubblized) PCCP. There have been instances where the MHB and Z-grid roller apparently rubblized the pavement, only to find that the section was too weak to support paving equipment and construction traffic (Chapter 3). The U.S. Air Force has guidance on proof rolling in Engineering Technical Letter 97-3, *Base Course Proof Rolling Requirements*. Since the goal of proof rolling a rubblized pavement is to identify soft or yielding areas, and not to further compact the base layers, using rubber tires with the lower pressure of 125 psi is appropriate (versus 150 psi). Selecting too heavy or too many rollers can cause excessive distortion on sections that are structurally adequate. More discussion on proof rolling considerations for thin pavement structures is provided in Sections 3.3.3 and 3.3.4 of this report.

6.5   Relief Trenches

When an RPB is used, the PCCP will expand laterally and <u>may</u> necessitate that relief trenches be cut into the PCCP. The need of a relief trench is realized when a slab is no longer taking impact energy and fracturing the slab. Wider features, thicker slabs and smaller RPB machines are more likely to require relief trenches. Existing joints and cracks may accommodate the fractured slab expansion. Drainage and isolation cuts can also serve as relief trenches. Relief trenches are rarely needed on highways because highways are not as wide as airfield features. Relief trenches are also mentioned in Section 2.5.4. The MHB machine does not require relief trenches.

Since it is common that relief trenches will not be required with airfield rubblization, a specification should simply include a statement that allows the use of relief trenches as the contractor deems appropriate.

6.6   Construction Sequence

The general approach to rubblization is typically as follows:
- Install/improve subsurface drainage
- Shoulder rehabilitation (if necessary)
- Remove existing HMA overlay
- Rubblize and roll PCCP
- HMA Paving

Section 2.5.4 of this report addresses how each of these steps are handled in the current P-215 and 01-09 specifications.

Drainage was discussed earlier in this chapter.

Shoulder rehabilitation is often performed on highway rubblization projects to repair the work done on the edge drain system and to allow for traffic to use it at least temporarily during construction phasing. Shoulder widening might also be necessary for airport pavement projects. If so, it will affect the details of the subsurface drainage system, and the shoulder structure will also include the overlay thickness placed over the rubblized PCCP. Shoulder rehabilitation on airfields will also likely have to deal with airfield lighting fixtures placed in the shoulders.

It is necessary to remove any existing HMA overlay by milling before attempting to fracture the PCCP. Any remaining material will cushion the impact of either type of rubblization equipment and likely cause a problem with achieving proper fracture. If traffic must use the

milled surface, the milled surface should be swept thoroughly to avoid bringing FOD onto an active part of the airfield. While most full-depth patches can remain in place prior to rubblization, it may be deemed necessary to remove and replace them. This should be accomplished using similar procedures as performed for repairing test pits. The ability to properly place and compact the replacement material should be a consideration when determining the size of the patch.

Finish grading of the rubblized surface, for smoothness or grade control, is not possible. Milling the PCCP prior to rubblization to correct grade or match finish grade has been accomplished on projects. One warning with milling PCCP is to not mill so much that the remaining PCCP is so thin it creates support issues (see Chapter 3).

Depending on the situation, it might be necessary to pre-fracture the PCCP or to cut "relief trenches" to facilitate rubblization. These issues should be resolved during the construction and evaluation of test strips. During the rubblization operation, it is important to assure that broken or loose materials cannot present a potential FOD hazard. This is noted within the current P-215, but it is something to review as the construction plan is detailed.

Applying a prime or tack coat on the rubblized surface is not recommended. A tack coat is only used on bound surfaces such as HMA, PCCP, or even crack and seated PCC. A prime coat can be applied on a dense, well-graded, tight aggregate surface. It is felt that the surface of a rubblized layer would generally not be well suited for a prime coat due to a lack of fines and variable texture.

6.7 Leveling Courses

A minimum lift thickness of 3 inches is recommended for the first lift of HMA placed over rubblized PCCP. The reason for this is to allow a reasonable opportunity to meet the in-place density requirements of the HMA specification. A consensus of paving contractors that have worked on rubblization projects have suggested this. A relatively thick lift helps reduce the impact that the loose rubblized surface condition has on successfully compacting the HMA. This is discussed further in Section 4.7. Aggregate leveling courses are also covered in Section 3.3.6 as a consideration on marginally stable (thin) pavements.

Another HMA paving issue that should be considered is that the paving contractor should NOT be permitted to use a windrow elevator to feed the hopper of the paving machine on the initial lift of HMA. This would risk disturbing/dislodging particles at the surface of the rubblized PCCP (see Section 3.3.8). Ideally, the paving contractor would use some form of material transfer vehicles (MTV) to charge the hopper of the paving machine in order to extend workability by conserving heat and minimizing segregation in the HMA layer. As mentioned at the end of Section 3.3.8, MTVs are typically very heavy and should be used with extreme caution on thin slabs that have been rubblized because they could induce rutting or shear failure.

6.8 Unstable Areas

Section 2.5.4 discusses unstable area patching. Repair material can be P-401 (HMA) or P-209 (crushed aggregate base). Quantities should be in terms of volume versus area since the

necessary depth of excavation may not be determined until removal of the weak subgrade. FAA specification P-152 can be referenced for subgrade excavation. A separate bid item should be included for full depth patching (Section 3.3.7). Full-depth patching efforts on the Kegelman Runway and Pratt Runway projects are discussed in Section 2.4.

6.9   Traffic Control

It is best to keep traffic away from the pavement being rubblized and overlayed until all work is completed, including all the HMA paving lifts. The minimum HMA overlay thicknesses from Section 4.7 should be adhered to. Heavy construction traffic loads directly on the rubblized surface (prior to HMA placement) should be minimized, especially on thin pavement structures (see Section 3.3.8).

6.10   Cost of Rubblization

The unit cost of rubblization (and rolling) will vary depending on many factors. These factors include availability of equipment, location of project, project size, work-site and work-hour restrictions, etc. The cost will also depend on the contractors' expectation as to how easy or difficult the PCCP will be to rubblize (i.e. slab thickness, steel, aggregate hardness, etc). Even the specification and expected enforcement of that specification can be factors to cost.

Table 6.1 comes from Antigo Construction Inc. and provides the average unit cost of rubblization through 2006 for 12 states. The states listed are those that have had several rubblization projects and report their bid data on a web site. Antigo has collected the low bidder's price data from these DOT web sites.

<div style="border:1px solid black;">

Weighted Average Bid Prices for Rubblizing Concrete Pavement*
(Low Bidder's Price Weighted by Quantity)

State	$ / SY	Total SY	Number of Projects	Timeframe
Iowa DOT	1.41	1,619,151	35	1997 - 2006
Ohio DOT	1.54	903,178	7	1998 - 2006
Indiana DOT	1.60	2,731,595	19	1999 - 2006
Minnesota DOT	1.62	485,564	5	1999 - 2006
Wisconsin DOT	1.63	5,190,526	78	1998 - 2006
Alabama DOT	1.78	1,745,920	16	1996 - 2006
North Carolina DOH	1.99	580,462	5	1998 - 2006
Illinois DOT	2.05	797,657	14	1996 - 2006
Utah DOT**	**2.11**	1,071,297	4	2002 - 2006
Michigan DOT Projects	2.18	2,712,196	52	1999 - 2006
Louisiana DOTD**	**2.57**	1,956,749	11	2002 - 2006
Arkansas State Hwy & Trans. Dept.**	**2.92**	6,954,807	***	1998 - 2006

* Data collected from various sources, see state sheets for details.
** **State rubblizing specification requires resonant breaker only.**
*** Annual report does not include number of projects.

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Table 6.1, Average Unit Cost of Rubblization by Individual States [Antigo, 2007]

As Antigo (who manufactures and operates the MHB) illustrates, the most expensive states are those that only allow the resonant pavement breaker (RPB) type in their specifications (Utah, Louisiana and Arkansas). These states feel the RPB provides a better product than the MHB in terms of greater aggregate interlock and less disturbance (or possible shearing) of the subgrade. This is why their specification is exclusive to the RPB. It is also worth noting that Michigan's average price is high relative to the other states that allow both the RPB and the MHB. This could be related to their strict enforcement of their specification regarding the steel being completely debonded, which has been discussed in Section 2.2.3 and in Chapter 5.

6.11 Concern of PCC with ASR

As part of this project, our research team was asked to look into a concern that a PCCP suffering from Alkali-Silica Reaction (ASR) could potentially expand after rubblization to cause future differential swelling on the pavement surface. This concern resulted in the publishing of recent guidance [AF ETL 06-07, 2007] that describes the rationale for the concern and guidance on performing an optional risk assessment procedure when recycling ASR-infected PCC back into a pavement structure. The ETL cites IPRF Report 03-5 on recycling PCC, which cautions engineers dealing with aggressive ASR in airfields to conduct a detailed benefit/risk analysis

when evaluating project options. It is emphasized that the risk assessment protocol described in ETL 06-07 is currently optional for engineers to use.

We initially had considerable communication with the U.S. Corp of Engineers Waterways Experiment Station in order to understand the basis of the concern. Our efforts then shifted to searching the literature and speaking to recognized rubblization experts regarding the concern that ASR-infected PCC could continue to react and cause expansion in a rubblized layer and subsequent surface roughness. We particularly looked at the performance and noted distresses of highway and airfield pavement that had utilized fractured slab technology on ASR-infected PCC. This section summarizes those findings and provides our comments on AF ETL 06-07.

6.11.1 Searching the Literature

We found no mention of this concern in our literature search, and no finding of any past projects that had experienced differential swelling or pavement roughness that should be attributed to an expanding ASR rubblized layer. Besides rubblization, we also looked at crack and seat (C/S) and break and seat (B/S). We still found no mention of the concern. To put this "found nothing" result into perspective, consider the following:

- Approximately 50 million square yards (SY) of PCC were rubblized in the U.S. in the past decade.
- C/S and B/S technology was used frequently prior to rubblization becoming popular in the '90s.
- Antigo has used C/S or B/S on 62.9 million SYs of pavement and rubblized 25.2 million SYs from 1982 to 2006 [Antigo, 2007].
- The PCS/Law Inc study conducted in 1991 and referenced in Chapter 2 cited roughly 500 highway projects in the U.S. that utilized either C/S, B/S or rubblization. The use of fractured slab technology has grown considerably since 1991 on both highways and airfields.
- ASR, D-cracking and other material-related distresses are often cited in the highway industry as problems for which rubblization is an appropriate rehabilitation solution.

There were several C/S or rubblization projects where ASR-infected PCC was documented beforehand and whose performance has been monitored.
- The Colorado I-76 rubblization project in 2000 and discussed in Chapter 2 had severe ASR. The construction report [CDOT, 2000] states the severe ASR was the major reason rubblization was selected for this heavily trafficked section. A recent follow-up report stated that performance six years later was good with no problems related to rubblization [LaForce for CDOT, 2006].
- The authors of this report recall three airfield rubblization projects that had ASR-infected PCC: Niagara Falls Runway in 2002, Buffalo Taxiway D in 2003 and Buffalo Taxiway A in 2005. No performance problems have been noted that would be attributed to an expanding PCC layer.
- The authors of this report recall two airfield B/S projects that had ASR in the PCC: Memphis Taxiway N in 1995 and Harrisburg International Airport Runway in 1996. Performance on the Harrisburg Runway has been outstanding. The Memphis Taxiway

was reconstructed two years ago, 10 years after the B/S project (designed for a 7-year life). There were two localized failures after those 10-years, neither related to the C/S.

Additional findings include:
- The consensus opinion from the "rubblization experts" we spoke to was that the void space created with the rubblization would accommodate any potential swelling caused by the ASR.
- One possible issue with fracturing severe ASR-infected PCC was not getting consistent breakage at the bottom of the slab because some of the fracture energy was absorbed at the top. This has not led to any actual construction or performance problems.
- Besides swelling, the other potential adverse effect stated in AF ETL 06-07 from on-going ASR is the loss of strength. As covered in Chapter 4, we found four separate projects where the modulus of the rubblized layer was monitored over time. In all cases, the modulus appeared to increase rather than decease, albeit not on a regular basis.

Given the vast amount of pavements that have been rubblized or crack/break and seated over the past 25 years, and given that ASR is not that uncommon, we believe it's fair to say that a significantly large amount of ASR-infected PCC has been rubblized or crack/break and seated. Combine this with the fact that we found no mention of these ASR concerns in our literature search efforts; it is our opinion that there is an extremely low probability that a rubblized ASR-infected PCC layer would expand to cause future differential swelling on the pavement surface.

6.11.2 Comments on AF ETL 06-07

In the context of the risk assessment protocol described in AF ETL 06-07, we believe the probability rating of "Unlikely" is much more appropriate versus the rating of "Seldom." The ETL suggests the rating of "Seldom" unless there are site-specific adjustment factors to be considered.

We disagree with the statement contained in Section 7.6.6 of the ETL that "ASR and sulfate attack reactions are in the same order of magnitude for volume change." This statement is the basis of the claim that secondary chemical reactions can cause dramatic failures such as those experienced at Holloman AFB. We understand the concerns related to sulfate heave. This is typically where a calcium-based stabilizer is used to treat materials high in sulfates where large volumes of water are readily available. The development of ettringite requires the incorporation of 26 moles of water to the crystalline structure. The volume change in this reaction can be dramatic as experienced at Holloman and many other locations around the world. The sulfate heave problem can be simulated in the laboratory where 3D volumetric swells in excess of 50% can be observed over short periods. ETL 06-07 then points out "classical testing studies of ASR in concrete have examined growth of laboratory specimens in terms of a few tenths of a percent expansion or less." This statement contradicts the "same order of magnitude" statement. Even with the additional surface area introduced by the rubblization process, we believe the "same order of magnitude" assumption is invalid.

In ETL 06-07, the descriptions and photos of the experiences cited (which are intended to justify the ASR concern) fall into two categories. They are:
1. "sulfate attack on recycled PCC, sulfate attack on stabilized materials and various waste products that underwent volume change within the pavement structure. These were not

ASR related, but they do illustrate what may happen with unexpected volume change within the pavement structure" (section 8.3, Figures 1 and 2 of the ETL). The key words here are **"these were not ASR related"**. The previous paragraph here points out major differences between sulfate attack reactions and ASR reactions.

2. ASR related symptoms as photographed in Figures 4 thru 7 of the ETL. Figures 4 and 7 are photos of minor ASR cracking with no swelling effects. Figures 5 and 6 are photos of PCC pavements with highly reactive ASR that resulted in significant collective horizontal displacement of the PCC edges. However, these photos only illustrate the resulting horizontal collective strain of many adjacent reacting PCC slabs (that is likely hundreds of feet wide), and not the vertical swelling (strain) of a single underlying PCC layer (that is between 6 to 24 inches thick) as hypothesized in the ETL. Material strain over hundreds of feet cannot be compared to material strain over 0.5 to 2 feet.

The experiences and photos cited in ETL 06-07 do not provide any reasonable likelihood of noticeable vertical swelling from a recycled PCC layer with ASR.

In conclusion, we do not believe the risk assessment protocol described in AF ETL 06-07 is necessary because of the lack of evidence to suggest the perceived potential problem has or will ever occur. There have been more than 25 years of performance history using fractured slab technology without any indication of such a feared occurrence. Due to the concerns, we believe there should be long-term monitoring of projects where either recycled or fractured ASR-infected PCC remained as a base layer. Additional test cases could also be constructed and monitored. We would also like to see the development of a lab swell test that might provide useful information on this concern.

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