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Engineering Properties of Resin Modified Pavement (RMP) for Mechanistic Design

Gary Lee Anderton

March 2000

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Preface

The research project documented in this report was sponsored by the U.S. Army Engineer Research and Development Center (ERDC). The project was funded by the ERDC Laboratory Discretionary Research and Development program.

The research was conducted by personnel of the Airfields and Pavements Division (APD), Geotechnical Laboratory (GL), ERDC, Vicksburg, MS, during the period October 1995 through September 1997. Dr. Gary L. Anderton was the Principal Investigator of the project and the author of this report. Other ERDC personnel involved in some of the laboratory tests include Messrs. Tommy Carr, Roosevelt Felix, Rogers Graham, Brian Green, Layne Logue, Herb McKnight, Joey Simmons, and Ms. August Giffin. Professor Carl Monismith and his staff at the University of California-Berkeley also participated in the laboratory analysis by conducting beam fatigue tests. This research was conducted and the report was written by Dr. Anderton in partial fulfillment of the requirements for the degree of Doctor of Philosophy at the University of Texas at Austin.

The project was conducted under the general supervision of Dr. Michael J. O'Connor, Acting Director, GL. Direct supervision was provided by Dr. David W. Pittman, Chief, APD.

At the time of publication of this report, Dr. Lewis E. Link was Acting Director of ERDC, and COL Robin R. Cababa, EN, was Commander.

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Chapter 1: Introduction

In this chapter, the reader is introduced to the basic concept of resin modified pavement (RMP). A detailed description of this new pavement surfacing is provided to establish its historical development and the current state of practice. A discussion of the research objectives, scope, and approach is also provided to relate the goals of this study to the existing state of practice and its shortcomings.

DESCRIPTION OF RMP

This section provides a detailed description of the resin modified pavement in terms of its material components, construction techniques, areas of application, structural design, costs, and history. This description provides the background for the research needs addressed in this study.

Materials and Construction

Resin modified pavement (RMP) is a composite pavement surfacing that uses a unique combination of asphalt concrete (AC) and portland cement concrete (PCC) materials in the same layer. The RMP layer is generally described as an open-graded asphalt concrete mixture containing 25- to 35-percent air voids which are filled with a resin modified portland cement grout. The open-graded asphalt concrete mixture and resin modified portland cement grout are produced and placed separately. The RMP is typically a 50-mm-thick layer placed on top of a flexible pavement substructure when newly-constructed. This same thickness may be placed on existing flexible or rigid pavement structures as well.

The open-graded asphalt concrete mixture is designed to be the initial "skeleton" of the RMP. A coarse aggregate gradation with very few fines is used along with a relatively low asphalt cement content (typically 3.5 to 4.5 percent by total weight) to produce 25- to 35-percent air voids in the mix after construction.

The open-graded asphalt concrete mixture can be produced in either a conventional batch plant or drum-mix plant and is placed with typical AC paving equipment (Figure 1.1). After placing, the open-graded asphalt concrete material is smoothed over with a minimal number of passes from a small (3-tonne maximum) steel-wheel roller (Figure 1.2).

The resin modified cement grout is composed of fly ash, silica sand, portland cement, water, and a cross polymer resin additive. The resin additive is generally composed of five parts water, two parts cross polymer resin of styrene and butadiene, and one part water reducing agent. The grout water-cement ratio is between 0.65 and 0.75, giving the grout a very fluid consistency. The cement grout material can be produced in a conventional concrete batch plant or a small portable mixer. After the asphalt mixture has cooled, the grout is poured onto the open-graded asphalt material and squeegeed over the surface (Figure 1.3). Most of the pore spaces in the open-graded asphalt concrete are filled with grout upon initial application by the forces of gravity (Figure 1.4). Several passes of the 3-tonne steel-wheel roller in the vibratory mode are then used to ensure full grout penetration into all accessible void spaces (Figure 1.5). This process of grout application and vibration may be repeated until all voids are filled with grout, which is identified when the open-graded asphalt material does not absorb any additional grout during the vibratory roller passes.

Depending upon the specific traffic needs, the freshly grouted surface may be hand broomed or mechanically textured to improve skid resistance. Usually, removal of excess grout using squeegees provides a suitable rough surface texture, which is somewhat similar in appearance to an exposed-aggregate concrete surfacing (Figure 1.6). Spray-on curing compounds, typical to the PCC industry, are generally used for short-term curing. The new RMP surfacing usually achieves full strength in 28 days, but it may be opened to pedestrian traffic in 24 hours and light automobile traffic in 3 days.

No joints are required to be cut in the RMP surfacing, unless the RMP is used to overlay existing jointed PCC. In this instance, joints are usually cut in the finished RMP surfacing to trace the underlying PCC joints in order to minimize reflective cracking. Also, perimeter joints between RMP and any adjacent PCC pavements are recommended, as these two pavement surfacings are thought to differ in their expansion and contraction rates during significant temperature changes. All joints cut in RMP are filled with asphalt-compatible joint sealant materials.

Areas of Application

RMP may be used in new pavement construction or in the rehabilitation of existing pavement structures. RMP is typically used as a low-cost alternative to a PCC rigid pavement or as a means of improving the pavement performance over an AC-surfaced flexible pavement. Field experience indicates that RMP may be used in practically any environmental conditions.

The RMP process has been used in a variety of applications on the international market, including airport and vehicular pavements, industrial and warehouse floorings, fuel depots and commercial gasoline stations, city plazas and malls, railway stations, and port facilities. Since its first commercial application in the United States in 1987, RMP has been used mostly on airport and airfield pavement projects. A listing of the known RMP projects in the United States is given in Table 1.1.

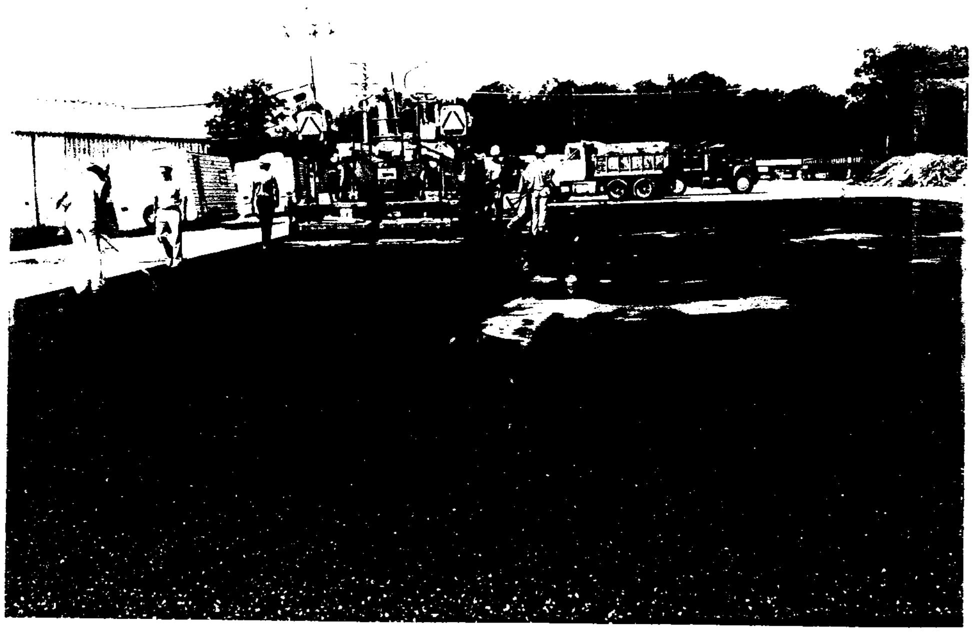

Figure 1.1: Typical view of placing open-graded asphalt concrete with asphalt paver

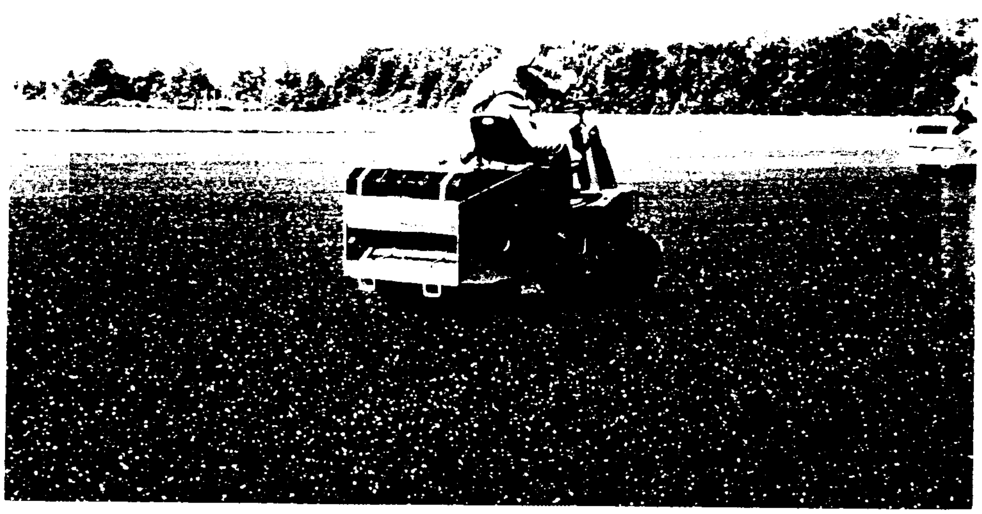

Figure 1.2: Typical view of rolling open-graded asphalt concrete with steel-wheel roller

Figure 1.3: Typical view of applying grout to open-graded asphalt concrete surface

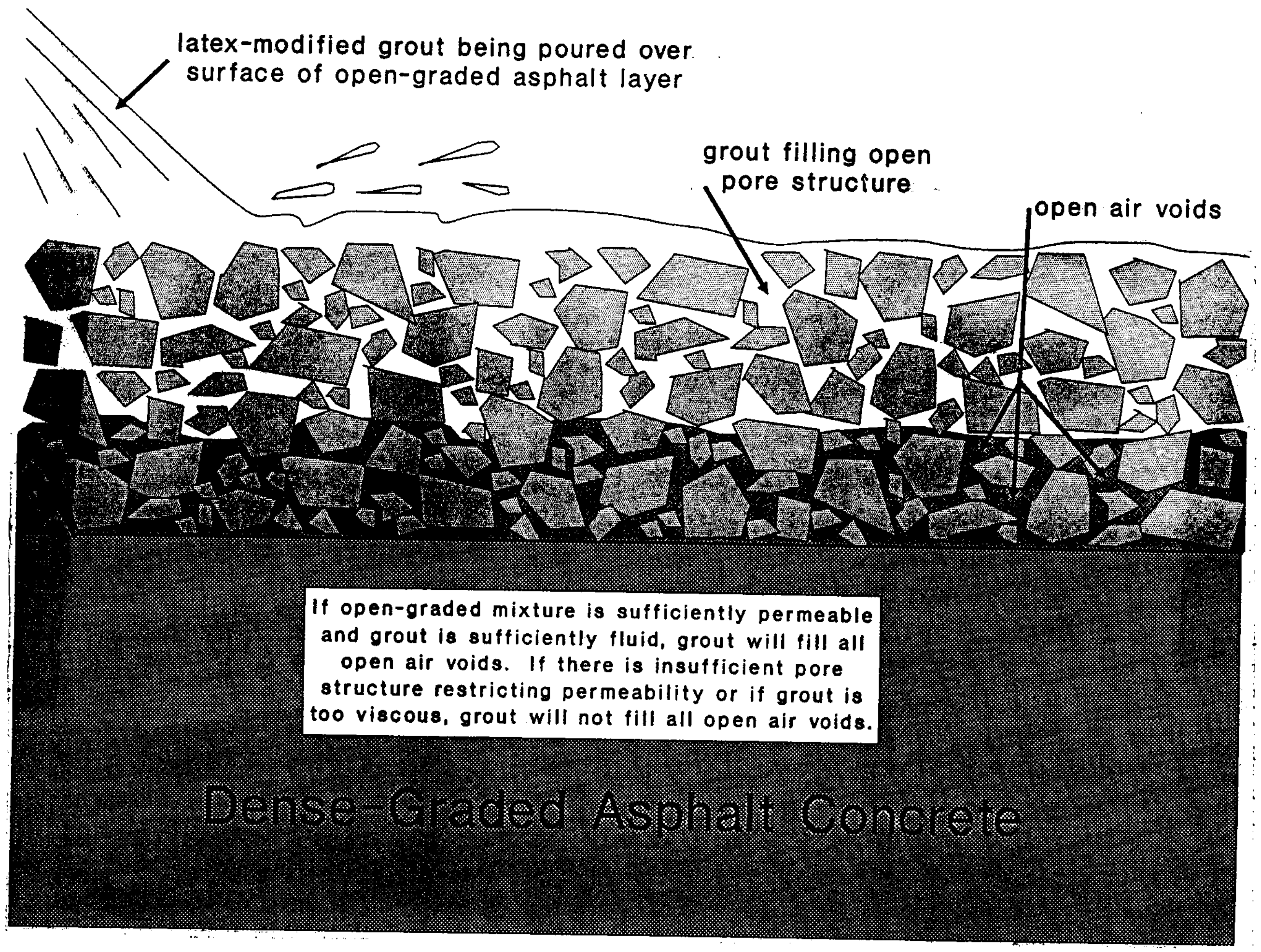

Figure 1.4: Cross-section of RMP as grout fills internal air voids

Figure 1.5: Typical view of steel-wheel roller vibrating grout into open-graded asphalt concrete layer

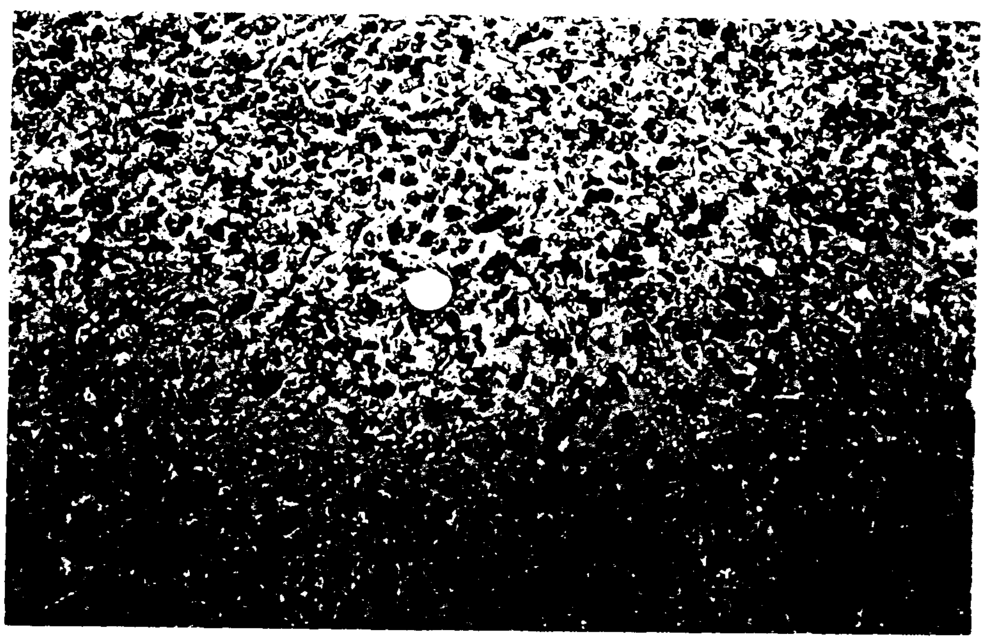

Figure 1.6: Typical appearance of resin modified pavement surfacing

Table 1.1: RMP Project Locations in the United States		
Location	Area (m^2)	Construction Date
Newark Airport, NJ (Aircraft Apron)	420	May 1987
Springfield, VA (GSA Parking Lot)	1670	Oct 1988
Vicksburg, MS (WES Test Section)	835	Aug 1989
Orange County, CA (Aircraft Taxiway)	8350	Oct 1990
Tampa Intl. Airport, FL (Aircraft Apron)	3350	Jan 1991
Miami Intl. Airport, FL (Aircraft Cargo Apron)	3350	Jan 1991
McChord AFB, WA (Loading Facilities) (Aircraft Apron)	8350 6600	Aug 1991 Apr 1996
Concord, CA (Port Facilities)	125000	1992, 1993, 1995, 1996
Fort Campbell, KY (Aircraft Apron)	6250	Aug 1992
Malmstrom AFB, MT (Fuel Storage Areas)	10835	Jun 1993
Fort Belvoir, VA (Loading Facilities)	8350	Jun 1994
Pope AFB, NC (Aircraft Aprons)	29170	Jun 1994
Altus AFB, OK (Aircraft Taxiway)	8000	Jun 1995
Johnstown ARC, PA (Aircraft Apron)	60000	Aug 1996
Aberdeen PG, MD (Tank Range Road)	1625	Sep 1996
Fort Jackson, SC (Tank Washrack)	5750	Sep 1996

Structural Design

The current practice for designing the RMP layer thicknesses involves a simple adaptation of whatever flexible pavement design method a particular agency is using. The pavement is designed as if it were a typical dense-graded AC-surfaced pavement and then the top 50-mm of AC is substituted with an equal thickness of RMP. This empirical design method is presumed to render a very conservative thickness profile in terms of strength and durability as the RMP material is thought to be much stiffer and resistant to traffic-induced damage when compared to traditional dense-graded asphalt concrete.

Costs

The cost of a 50-mm-thick RMP layer is currently about $9.60 to 19.20 per square meter ($8 to 16 per square yard) as compared to a typical cost of $3.60 to 6.00 per square meter ($3 to 5 per square yard) for a 50-mm-thick layer of dense-graded asphalt concrete. The initial cost of a full-depth RMP design is generally 50 to 80 percent higher than a comparable asphalt concrete design when considering a heavy-duty pavement. A more important cost comparison is between the RMP design and the rigid pavement design since the RMP is usually used as a cost saving alternative to the standard PCC pavement. In the case of a standard military heavy-duty pavement application, the RMP design should be 30 to 60 percent less in initial cost than a comparable PCC pavement design. In many circumstances, the RMP also provides cost savings from reduced or eliminated maintenance efforts when compared to other pavement surfacing alternatives.

History of RMP

The RMP process was developed in France in the 1960's as a fuel-and abrasion-resistant surfacing material. The RMP process, or Salviacim process as it is known in Europe, was developed by the French construction company Jean

Lefebvre Enterprises as a cost-effective alternative to PCC (Roffe 1989a). RMP has been successfully marketed throughout France as a pavement and flooring material in numerous applications. By 1990, Jean Lefebvre Enterprises had successfully placed over 8.3 million square meters (10 million square yards) of Salviacim pavement in France (Jean Lefebvre Enterprise 1990). Today, RMP is an accepted standard paving material throughout France.

Soon after the RMP process became successful in France, its use in other countries began to grow. In the 1970's and 1980's, RMP usage spread throughout Europe and into several countries in Africa, the South Pacific, the Far East, and North America (Ahlrich and Anderton 1991a). Twenty-five countries around the world had documented experience with RMP by 1990 (Jean Lefebvre Enterprise 1990). These countries and their respective amounts of RMP constructed up to the year 1990 are listed in Table 1.2.

The earliest documented experience with RMP in the United States occurred in the mid-1970's when the U.S. Army Engineer Waterways Experiment Station (WES) conducted limited evaluations of an RMP test section constructed in Vicksburg, MS (Rone 1976). The study was conducted to evaluate the effectiveness of the new surfacing material to resist damage caused by fuel and oil spillage and abrasion from tracked vehicles. The evaluation results indicated that the effectiveness of the RMP was very construction sensitive, and if all phases of design and construction were not performed correctly, the RMP process would not work.

In 1987, the U.S. Army Corps of Engineers tasked WES to reevaluate the RMP process for potential military pavement applications, since the field experiences in Europe continued to be positive and improved materials and construction procedures had been reported. WES engineers conducted literature

Table 1.2: Countries with Resin Modified Pavements	
Country	RMP In-Place ($m^2 \times 10^3$)
France	8356
Portugal	962
Japan	602
Great Britain	307
United States	288
Germany	282
Denmark	230
Sweden	221
Norway	188
Italy	183
Finland	148
Belgium	119
Switzerland	117
Saudi Arabia	100
Holland	82
Morocco	66
Ivory Coast	55
South Africa	34
Bahamas	29
Spain	22
Luxemburg	16
New Caledonia	15
Austria	14
Senegal	6
Tahiti	5
Note: Pavement area for United States based on 1996 data. All other countries based on 1990 data.	

reviews, made site evaluations in France, Great Britain, and Australia, and constructed and evaluated a new test section at WES (Ahlrich and Anderton 1991b). The results of this evaluation were favorable, prompting pilot projects at several military installations in the following years. The Federal Aviation Administration, also eager to develop an alternative paving material technology, used the positive WES experiences and preliminary guidance to construct several pilot projects at commercial airports (Ahlrich and Anderton 1993).

The RMP construction experience in the United States to date, as previously detailed in Table 1.1, can be divided into two distinct 5-year periods. The first time frame, from 1987 through early-1991, generally includes smaller-scale test sections and the FAA pilot projects. The second time frame, from mid-1991 through 1996, is when the larger, full-scale projects were constructed at nine military sites and one private industry site. Some of these full-scale projects will be discussed later in this report.

The RMP concept is not as widely-known in the United States as it is in France and many other countries. Continued growth in the number of RMP projects constructed and continual monitoring and reporting on the existing field applications should increase the pavement industry's awareness of this promising new technology. Today, the RMP process is recommended as an alternative pavement surfacing material by those agencies responsible for the bulk of the projects constructed in the United States; namely, the U.S. Army, the U.S. Air Force, and the Federal Aviation Administration (FAA).

SUMMARY

The current state-of-practice for RMP is empirically-based upon the limited experiences of the French paving company who originally developed the paving concept and those of a small number of agencies in the United States who have used RMP in recent years. Although field experiences in the United States have been limited, early performance has shown great promise for expanded future use. Further development of this new paving technology requires a better understanding of its fundamental engineering properties and a mechanistic approach to structural design to replace the current empirical method.

RESEARCH OBJECTIVES

The two primary objectives of this research are:

1. Determine the engineering properties of RMP relating to field performance, which are heretofore unknown;
2. Develop a mechanistic pavement design and modeling technique to allow for fundamentally-sound RMP thickness designs and performance predictions.

RESEARCH SCOPE

The scope of this research includes several specific tasks:

1. Conduct a comprehensive literature review on the history and previous research relating to RMP and comparable pavement technologies;
2. Conduct site inspections of past RMP field applications to determine critical pavement performance issues and existing failure modes;
3. Conduct site inspections of new RMP construction projects to determine critical construction issues and collect RMP samples for subsequent laboratory evaluations;
4. Gather laboratory materials and conduct RMP mix designs for production of standardized RMP laboratory test samples;
5. Conduct laboratory tests to determine engineering properties of RMP relating to critical performance-related qualities;
6. Develop a suitable RMP thickness design and performance modeling technique using appropriate engineering properties determined from standard laboratory-produced samples and field samples.

RESEARCH APPROACH

The approach to and sequence of the execution of the above tasks are illustrated in the flowchart shown in Figure 1.7.

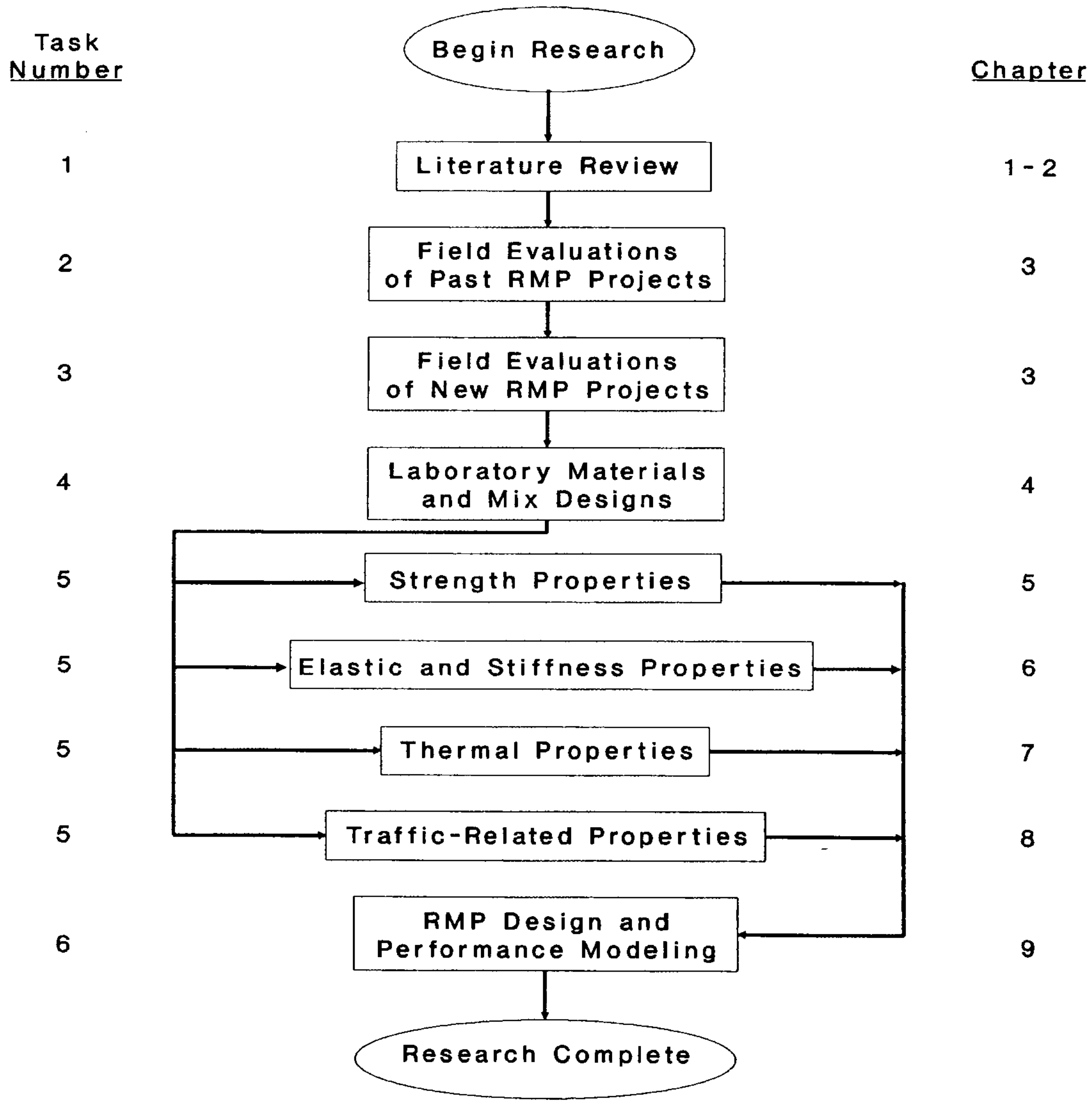

Figure 1.7: Flow chart of research approach

Chapter 2: Review Of Literature

Since RMP has a relatively short and isolated history in the United States, the literature on this subject is quite limited. In addition, most of the literature produced outside of the United States is typically directed at marketing the technology, rather than providing any detailed engineering data. This lack of technical information on RMP points out the importance of the research described in this report. The following sections of this chapter describe the available literature on RMP, as well as a review of a similar pavement surfacing technology known as Ultra-Thin Whitetopping.

DESIGN AND APPLICATIONS

Roffe authored two marketing publications for Jean Lefebvre Enterprises in 1989 (Roffe 1989a and Roffe 1989b), which offered general descriptions of the Salviacim paving process. These publications contained fundamental mix design guidance, such as recommended open-graded aggregate gradations and grout formulations, but offered very little information on thickness design and engineering properties. This lack of technical publications in France, where the RMP technology began, may be explained by the practice in many European countries where pavement contractors take on the responsibilities of developing, implementing, and even maintaining new pavement technologies. Large contractors in these countries protect their unique technologies through company secrecy or trade patents; therefore, the publications produced by these companies are few in number and rarely contain much technical detail. The Roffe publications and several unpublished confidential internal documents produced by Jean Lefebvre Enterprises provided the only background data for the early WES research on RMP during the 1980's.

Rone reported on the earliest evaluations of the Salviacim pavement process in the United States in 1976 (Rone 1976). This investigation was conducted at WES for the U.S. Army to determine the pavement's ability to resist damage from fuel and oil spillage and from the abrasive action of tracked vehicles. The guidance provided to WES engineers at that time allowed for aggregate gradations resulting in air void contents of 15 to 25 percent, which is generally 10 percent less than the current gradations typically produce. The lower air void content mixture used in the 1976 WES study was the major reason for the test section failure which highlighted the study results. Because the portland cement grout failed to fully penetrate the open-graded asphalt mixture, the weakened pavement surfacing cracked under initial trafficking and allowed the fuel and oil spills to penetrate down to the open pore spaces just beneath the surface grout. All of this led to a rapid pavement failure and an ultimate recommendation to discontinue future research until improved mix proportioning guidance was developed.

In 1987, the U.S. Army Corps of Engineers tasked pavements researchers at WES to re-evaluate the Salviacim process, as reports from military engineers in Europe indicated a great deal of success with this technology in numerous countries. WES researchers reported on site visits made in France and Great Britain and on a test section which was constructed and trafficked at WES (Ahlrich and Anderton 1991a and 1991b). New guidance obtained from Jean Lefebvre Enterprises recommended a more open grading in the aggregates, resulting in air void contents in the 25 to 35 percent range. The test section was successfully constructed and was deemed to be fully resistant to damage from heavy fuel and oil spills and from M1 and M60 armored tanks.

The favorable results of the second WES test section evaluation led to several military projects in the following years, including one which was documented in 1993 (Anderton and Ahlrich 1993). Numerous airfield pavement areas at Fort Campbell Army Airfield, Kentucky were reconstructed in 1992. One pavement area which was reconstructed using an RMP surfacing was a 6250 m^2 jointed PCC warm-up apron located at the end of the main runway. The existing PCC had significant cracking throughout the apron area and instead of removing it before reconstructing, 50 to 150 mm of dense-graded asphalt concrete plus 50 mm of RMP were placed on top of the existing surface to match new grade requirements. WES engineers noted two valuable lessons from the construction experience: (1) Uniformity in grout materials is essential. During construction at Fort Campbell, the contractor attempted to substitute a coarser sand gradation in the grout when the approved sand ran out. The resulting grout would not effectively penetrate the open-graded asphalt concrete, causing a work stoppage and unsightly repair on the warm-up apron. (2) All excess grout should be removed when skid resistance is an issue. Several isolated areas having excess surface grout were noted to have unsuitable skid resistance after curing, which required shot-blasting to roughen the slick surfacing.

Soon after the preliminary guidance provided by the second WES test section was published, the Federal Aviation Administration (FAA) authorized the construction of three pilot projects using RMP to reconstruct airport pavements at three commercial airports. WES engineers evaluated these FAA pilot projects in 1990 and 1991, and reported their findings in 1993 (Ahlrich and Anderton 1993). The authors concluded that the FAA had been successful in applying the RMP technology to certain airport pavement applications, including parking aprons and low-speed taxiways, and that the existing design and construction guidance was suitable for the FAA needs.

The current state-of-the-art practice for RMP is supported by the aforementioned documentation and three U.S. Army Corps of Engineers documents aimed at aiding the pavement designer and specifier. A U.S. Army Corps of Engineers Engineer Technical Letter (Headquarters, Department of the Army 1997a) provides guidance on RMP mix design and quality control test procedures (Appendix A). A Corps of Engineers Guide Specification (Headquarters, Department of the Army 1997b) provides guidance to the RMP contract specifier and contractor on proper design details and construction procedures (Appendix B). WES has published a user's guide report on RMP which summarizes the recommended applications, benefits, limitations, costs, design, and construction techniques (Anderton 1996). The guidance provided in these three documents can be described as the culmination of ten years of trial-and-error experiences from the WES research laboratories and from the first construction projects.

ENGINEERING PROPERTIES

Some of the earliest recorded investigations into the engineering properties of Salviacim were conducted for a British construction company known as TARMAC. Blight summarized much of this early research in a 1984 report to the South African subsidiary of the TARMAC company (Blight 1984). Blight conducted his studies at the University of Witwatersand in Johannesburg, South Africa where several laboratory tests were used to supplement the existing stiffness and fatigue data. Blight's tests included evaluations of Marshall stability and flow, resistance to impact loads, static creep, heat resistance, air and water permeability, and resistance to chemical attack. These tests were mainly designed to determine

Salviacim's suitability as an industrial flooring material. Blight compared the Salviacim test properties to the properties of standard asphalt concrete and portland cement concrete and found Salviacim particularly resistant to heavy static and impact loads, creep, hot spillage, and fuel and chemical spillage.

Harry Stanger Laboratories conducted a laboratory evaluation for TARMAC on pavement samples taken from a 1986 Salviacim trial section constructed in Great Britain (TARMAC 1986). The Salviacim samples were tested for a variety of physical and chemical properties and compared to typical hot rolled asphalt and PCC. Test methods used followed the appropriate British standards (BS) guidelines. The Salviacim samples were found to be moderately sensitive to test temperature when measuring compressive strengths, and the compressive strengths were noted to fall in a range between the normal values for hot rolled asphalt and PCC. The skid resistance of Salviacim was found to be relatively low on the untrafficked areas when wet, but improved significantly after trafficking to a point fully acceptable to British pavement standards.

The RMP surfacing material was evaluated at Virginia Polytechnic Institute under contract to the Strategic Highway Research Program (SHRP) (Al-Qadi, et al 1993 and 1994). The objective of the SHRP research was to evaluate the mechanical properties of RMP and to determine its durability characteristics in relation to concrete bridge deck protection, repair, and rehabilitation. The mechanical properties of the RMP were evaluated with several tests including Marshall stability, indirect tensile strength, resilient modulus, and compressive strength. The durability characteristics of the RMP were evaluated by testing the water sensitivity (loss of strength upon soaking) and freeze-thaw properties. A summary of these test results is listed in Table 2.1. These data show that the RMP material has mechanical properties and durability characteristics equal to or better than a high quality hot mix asphalt (HMA).

Table 2.1: Summary of Laboratory Results from SHRP Evaluation		
Material Property	HMA	RMP
Marshall Stability (kN)	8.7	19.0
Indirect Tensile Strength (kPa)	715	985
Tensile Strength Ratio		
Water Sensitivity	0.87	0.72
Freeze-Thaw	0.70	0.66 - 0.89
Resilient Modulus (MPa)	2040	4937
Resilient Modulus Ratio		
Water Sensitivity	0.83	0.82
Freeze-Thaw	0.68	0.51-0.78
Compressive Strength (MPa)	1.2	5.5

ULTRA-THIN WHITETOPPING

Ulra-Thin Whitetopping (UTW) is the only documented pavement surfacing that is comparable to RMP in terms of layer thickness and applications. UTW is a process where a layer 50 to 100 mm thick of high strength portland cement concrete is placed over a subbase of milled asphalt concrete. A review of the literature on this subject is provided to summarize the important similarities and differences between UTW and RMP.

UTW is a recent innovation in the pavements industry that has evolved from two other PCC overlay technologies: full-depth whitetopping and fast-track whitetopping. Full-depth whitetopping, commonly referred to as simply "whitetopping," refers to the application of a concrete overlay on an existing asphalt concrete pavement with no attempt to provide any bonding between the two layers. The first documented whitetopping project was constructed in Terre

Haute, Indiana in 1918. Whitetopping has since grown in popularity, with a total of 178 projects constructed through 1992 on highways, county roads, streets, and airports (Packard 1996). Even with this success, there were always two main obstacles limiting the usage of whitetopping (Speakman and Scott 1996):

1. Length of curing time (usually 3 to 7 days before traffic can return to the pavement), and
2. Increase in surface elevations too high because of the combined thickness of the old pavement and the new concrete overlay.

Fast-track whitetopping was developed during the 1970's to address the whitetopping issue of curing time. Material suppliers, equipment manufacturers, contractors, engineers, and designers all over the United States combined efforts to create a portland cement concrete that could develop strength fast enough in an overlay situation to permit early return of traffic. Fast track concrete is usually designed to carry standard traffic loads within eight hours and heavy traffic within 24 hours after placement is completed. This early strength gain is typically achieved by using various combinations of high-early strength portland cement, low water-cement ratios, water reducers, and some type of fiber reinforcement. As with standard whitetopping, fast-track whitetopping construction usually makes no effort to provide enhanced bonding between the portland cement concrete overlay and the underlying asphalt concrete. The development of the fast-track whitetopping method addressed the issue of curing time, but the traditional overlay thicknesses used did not address the issue of the excessive increases in surface elevations.

During the early 1990's, UTW was developed to allow the whitetopping technology to be used in rehabilitation projects requiring both high early strength and thinner cross sections. Concrete overlay thicknesses from 50- to 100-mm are allowed by the addition of polypropylene fibers to enhance material strength and by milling the underlying asphalt concrete which provides a bonding mechanism between the two concrete layers. Since the UTW layer is bonded to the asphalt concrete layer, a single composite pavement layer is created which results in a much stronger surfacing material. It is well-known that bonded concrete overlays allow for thinner surfacings when compared to unbonded overlays, and it is this pavement engineering principal that allows for a much thinner overlay with the UTW method.

After the first UTW project was constructed in Louisville, Kentucky during 1991, at least 68 additional UTW projects were constructed through 1996 (Mack 1996). Forty-four of these projects were built in the United States, with twenty-one in Mexico, two in Sweden, and one in Canada. Of the nearly 638,000 m^2 of total UTW placed, Mexico has placed about 519,000 m^2. The two most significant UTW projects in the United States are at the Spirit of St. Louis Airport in St. Louis, Missouri, and on Iowa Highway 21 near Belle Plain, Iowa. The St. Louis project has electronic instrumentation embedded within the pavement surfacing to measure pavement response (Mowris 1995) and the Iowa project includes 64 different test sections (Speakman and Scott 1996). These two projects along with several others in the United States are currently being monitored to aid in the development of standardized thickness design, material design, and construction procedures.

UTW and RMP are similar in a number of ways. Both techniques involve thin, rigid surfacings over flexible pavement structures. Both are designed to provide improved performance over standard asphalt concrete surfacings. The most significant attractions for both the UTW and RMP technologies are that they are relatively easy to construct and they can provide significant savings in terms of first cost and maintenance costs as a pavement rehabilitation alternative.

There are many critical differences between the existing UTW and RMP technologies. Aside from the coarse stones, the UTW mortar generally consists of portland cement, coarse-graded natural sand, water, a high range water reducer, and the polypropylene fibers. Besides the coarse aggregates used in RMP, the grout consists of portland cement, fine-graded processed sand, much higher water contents, and an SBR latex rubber plasticizer. UTW requires tight joint spacing (0.3 m joint spacing per 25 mm of concrete depth); RMP requires no joints when overlaying asphalt concrete. UTW is designed for high early strength, allowing for heavy traffic within 24 hours of placement; RMP is typically not designed for high early strength, generally allowing for heavy traffic only after 14 to 28 days after grout placement. Construction experiences in Tennessee (Speakman and Scott 1996) have indicated a UTW unit cost of approximately $38.28 per square meter for a 75-mm thickness; RMP construction experiences throughout the United States indicate a current RMP unit cost range of approximately $9.60 to 19.20 per square meter for a 50-mm thickness. These cost data indicate that the unit cost of UTW is approximately two times higher than the unit cost of RMP.

Chapter 3: Field Evaluations

Site inspections of RMP construction projects were conducted to supplement this research study. Four existing RMP project sites were inspected before the laboratory test plans were finalized. After the laboratory testing had begun, two new RMP project sites were inspected as they were being constructed. The following sections of this chapter describe the site inspections of the pre-existing and new RMP projects, and the conclusions reached from all of the field evaluations.

SITE INSPECTIONS OF PAST PROJECTS

The actual work of this research study began with visual inspections of four existing military airfield RMP projects. These inspections were made so that the pavement distresses observed on existing RMP sites could be used to direct the laboratory testing towards the most critical failure modes. The four sites were selected to provide observations of RMP with different ages, pavement design conditions, traffic, and environmental conditions. The important findings from each of these field evaluations are presented in the following sections.

Malmstrom Air Force Base, Montana

An airfield fuel storage area at Malmstrom Air Force Base (AFB), Montana was inspected during September, 1995. At that time, the RMP surfacing at this site was two years and three months old. The site consisted of about 11,000 sq m of RMP with numerous fuel pumps and aircraft refueling tanker trucks stored throughout the area. Underlying areas of expansive clays made a somewhat flexible pavement structure more desirable and the potential fuel spillage required a fuel-resistant pavement surfacing. These were the main reasons for the use of RMP at this site. An overall view of the RMP site at Malmstrom AFB is shown in Figure 3.1.

The general condition of the RMP at Malmstrom AFB during the time of the site inspection was good. As expected, several pavement areas showed signs of significant fuel and oil spillage (Figure 3.2). However, no structural or surface damage was noted to have been caused by any fuel or oil spills.

Figure 3.1: Overall view of RMP-surfaced fuel storage yard at Malmstrom AFB

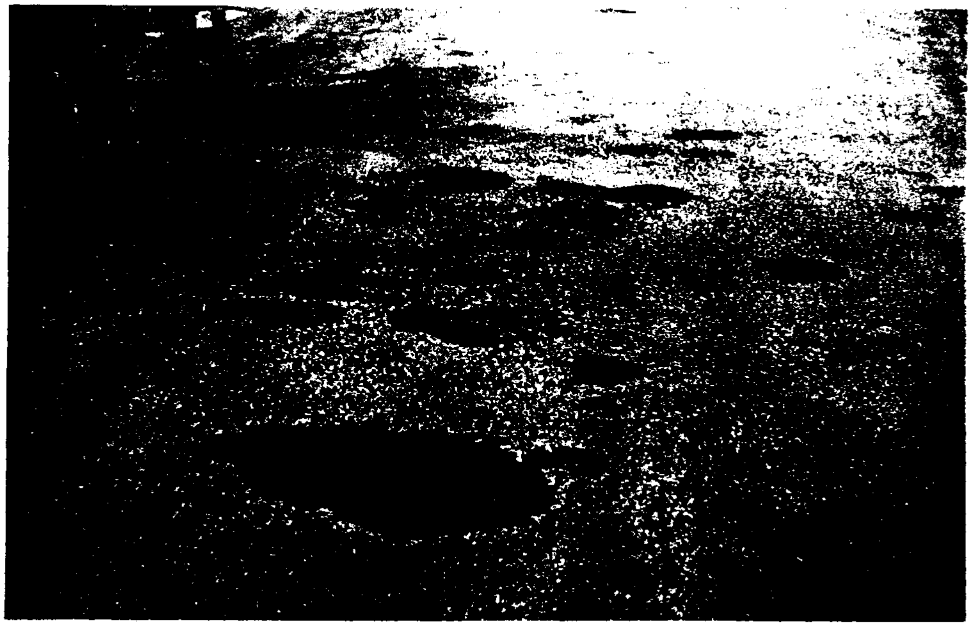

Figure 3.2: Fuel spillage on RMP at Malmstrom AFB fuel storage yard

There were three types of isolated pavement cracking identified during the inspection. A series of parallel and interconnecting cracks was found in one area where the pavement had heaved upwards (Figure 3.3), and construction records indicated that this area contained significant amounts of expansive clay material in the natural subgrade. Isolated areas of subsurface heaving were somewhat expected and if the RMP were able to maintain its structural integrity under these conditions, then it would have served its purpose probably better than a PCC surfacing could have under the same conditions.

The second type of pavement cracking found at Malmstrom AFB was in an isolated area where about 23 m of a longitudinal construction joint had opened up to a crack width of approximately 8 mm (Figure 3.4). There were no adjacent cracks running parallel to the joint crack and the material along the crack was not raveling. This indicated a mistake was made during construction by not getting a sufficient amount of grout into this particular joint area.

Figure 3.3: Isolated cracking and surface heaving in RMP caused by expansive clay subgrade at Malmstrom AFB

Figure 3.4: Small crack in longitudinal construction joint between lanes of RMP at Malmstrom AFB

The third type of crack found at this site was a transverse crack near the center of a 46-m-long and 3.6-m-wide roadway designed for one-way vehicular traffic (Figure 3.5). This crack ran across the full width of the roadway. The geometrical influences of a long and relatively narrow pavement area combined with the extremely cold winter temperatures at this site indicated that this crack could have been thermally-induced. There was only one crack of this type found on the roadway.

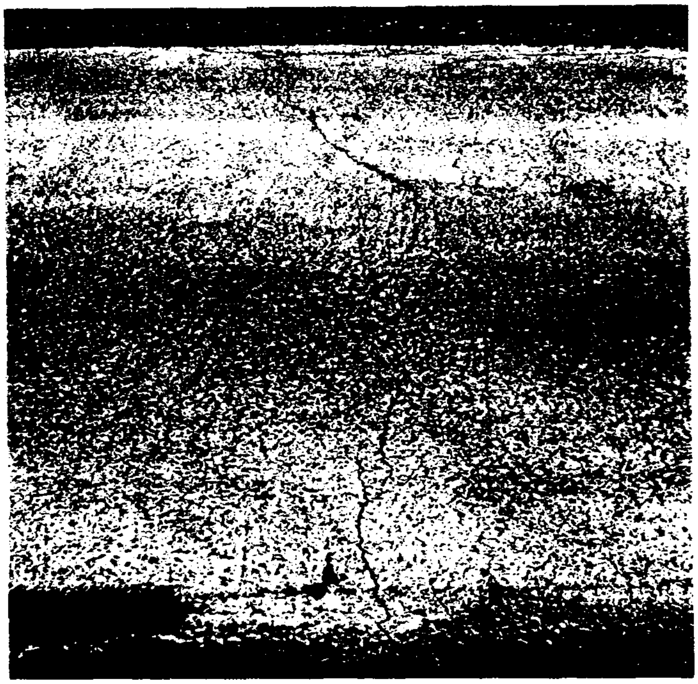

Figure 3.5: Possible thermal crack in single-lane RMP roadway at Malmstrom AFB

McChord Air Force Base, Washington

Two RMP-surfaced storage areas at McChord AFB, Washington were inspected during September, 1995. These pavements were approximately four years old at the time of this inspection. Each of these areas was used to load and unload air transportable containers and equipment, and each pavement area routinely carried heavy truck and forklift traffic. RMP was reportedly used at these sites to resist the heavy traffic and point loads while reducing construction efforts, first costs, and maintenance costs when compared to PCC. A typical view of these RMP applications at McChord AFB is shown in Figure 3.6.

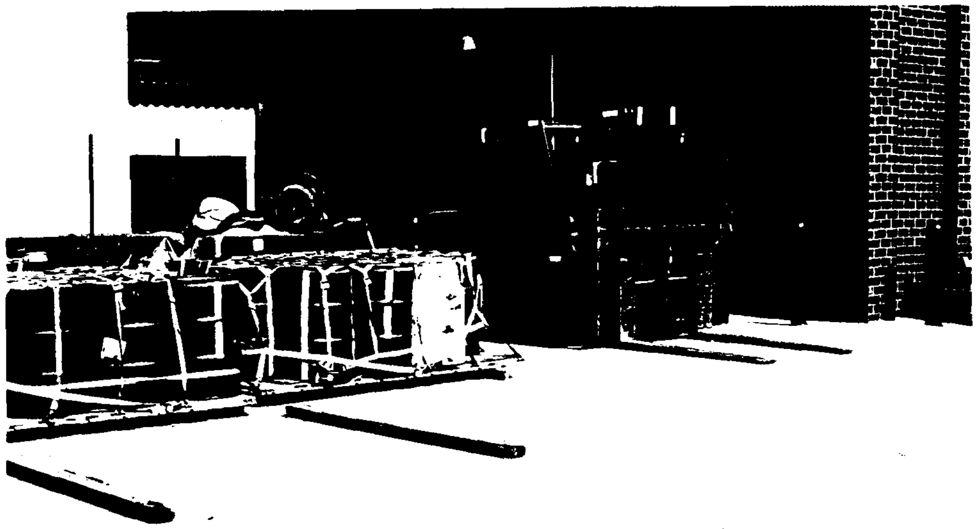

Figure 3.6: Forklift and air cargo on RMP at McChord AFB

The McChord AFB sites are the oldest military RMP projects in the United States, which made them important sites for evaluating durability issues. As it turned out, these sites appeared to be in the best condition of all existing sites inspected. The RMP at this site proved to be resistant to damage from heavy point loads and severe forklift traffic. Virtually, no cracks, ruts, or other surface damage were noted at this site.

Pope Air Force Base, North Carolina

Three adjacent RMP-surfaced aircraft parking aprons at Pope Air Force Base, North Carolina were inspected in November, 1995. These airfield apron pavements were about 17 months old at the time of this inspection, having been reconstructed during the May-June, 1994 time frame. The airfield aprons were designed to carry various fighter class aircraft and cargo aircraft loads up to the C-141 aircraft. It was noted by the airfield manager during this site inspection that several aircraft considerably larger than the C-141 design aircraft (specifically, the 747 and C-5A) had been using these aprons with no apparent problems. He also pointed out that several helicopters would routinely use one of the aircraft aprons, causing numerous fuel spills during normal operations but no apparent damage to the RMP surfacing.

What made the Pope AFB aprons an important site for these inspections was that three different reconstruction designs were used at the same site. U.S. Air Force engineers specifically used this strategy to help determine the most appropriate choice for future airfield pavement reconstruction projects involving the use of RMP. The three design strategies used at Pope AFB are shown in Figure 3.7 and are briefly described as follows: Snack Bar Apron involved milling 50 mm of existing asphalt concrete (AC) overlay from the jointed portland cement concrete (JPCC) pavement, cracking and seating the remaining 15- to 23-cm-thick JPCC slabs, and overlaying the cracked and seated JPCC pavement with 50 mm of AC binder course and 50 mm of RMP. Operations Apron involved milling 50 mm of tar rubber concrete overlay from the existing JPCC, and overlaying the remaining JPCC with 50 mm of AC binder course and 50 mm of RMP. Once the RMP had sufficiently cured, joints were cut on the Operations Apron pavements to trace the joints in the underlying JPCC and, thereby, control reflective cracking in the overlay. Hanger 6 Apron involved removing the existing 75 mm of AC surface course, base, subbase and subgrade to a depth of 56 cm, and replacement with 46 cm of crushed aggregate base material, 50 mm of AC binder course, and 50 mm of RMP.

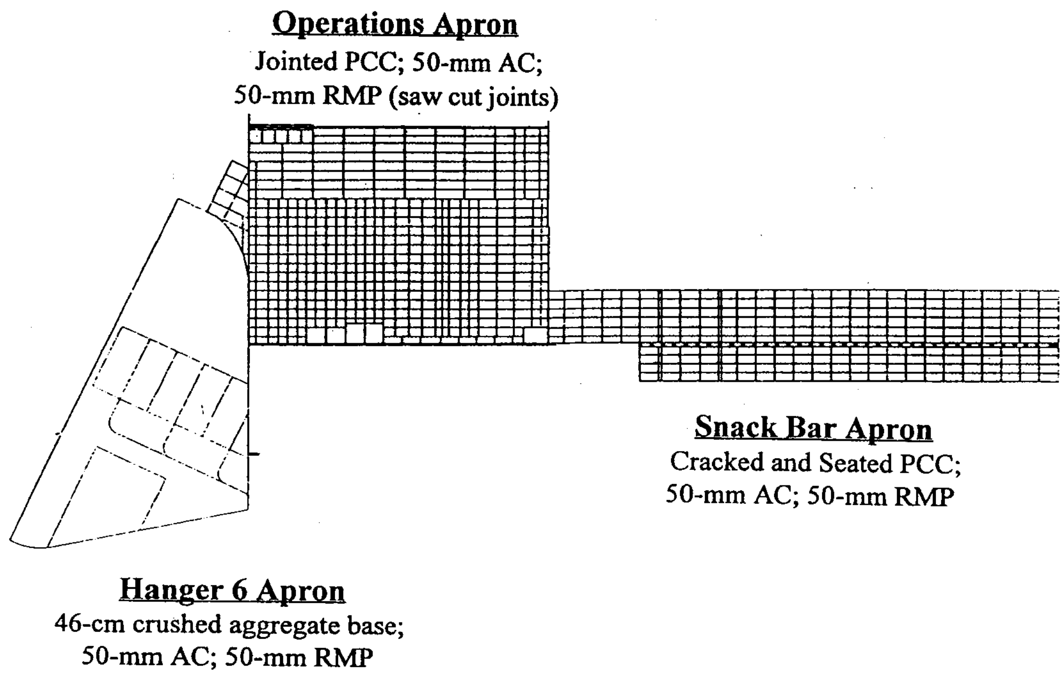

Figure 3.7: Layout of RMP parking aprons at Pope AFB

The Snack Bar Apron appeared to be in very good condition with only a few random hairline cracks evident. There was no evidence of reflective cracking from the underlying cracked and seated JPCC, nor was there evidence of any load-related or environmental damage. The surface texture on the Snack Bar Apron appeared to be uniform and in an ideal condition for optimum skid resistance. An overall view of the Snack Bar Apron is shown in Figure 3.8.

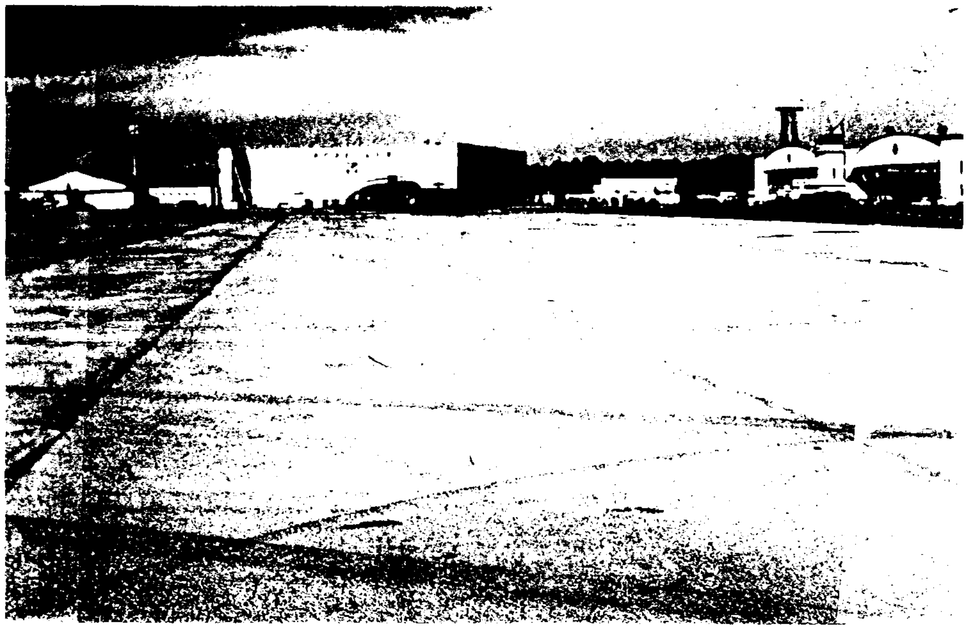

Figure 3.8: Overall view of RMP-surfaced Snack Bar Apron

The Operations Apron appeared to be in good condition with some minor reflective cracking evident (Figure 3.9). These cracks were noted to have probably resulted from existing mid-slab cracks in the underlying JPCC slabs. In addition to the reflective cracks, there were areas along the RMP and portland cement concrete (PCC) interface where some irregular cracking parallel to the interface was evident (Figure 3.10). These cracks near the RMP/PCC interface may have been caused by a partial bonding between the RMP and PCC left in place when the interface was saw cut to a 25 mm depth, leaving a 25 mm thickness of RMP bonded to the PCC. When environmental conditions caused the PCC slabs to move away from the RMP, a bonded condition could have resulted in the cracking of the RMP near this interface. Besides the reflective cracking caused by the underlying JPCC slabs and the cracking along the RMP/PCC interface, the overall condition of the Operations Apron appeared to be good. The surface texture on this apron appeared similar to that of the Snack Bar Apron and in good condition.

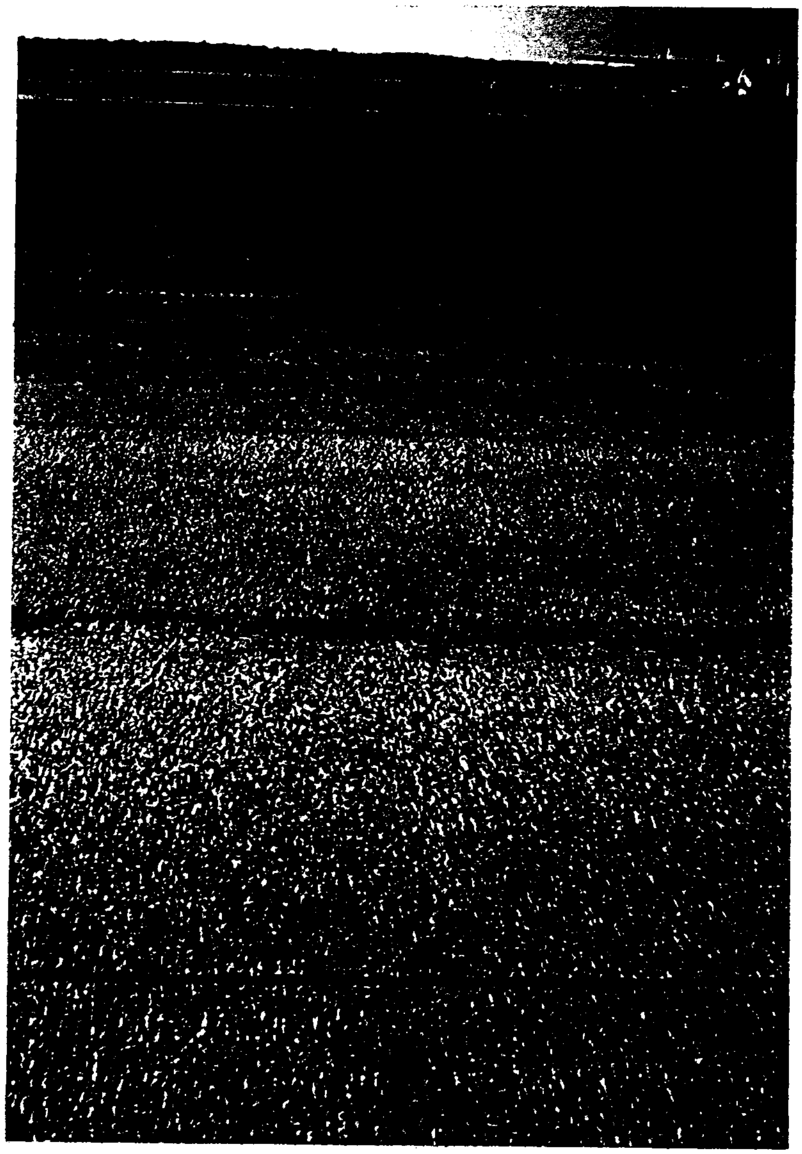

Figure 3.9: Minor reflective cracking in RMP overlay on Operations Apron

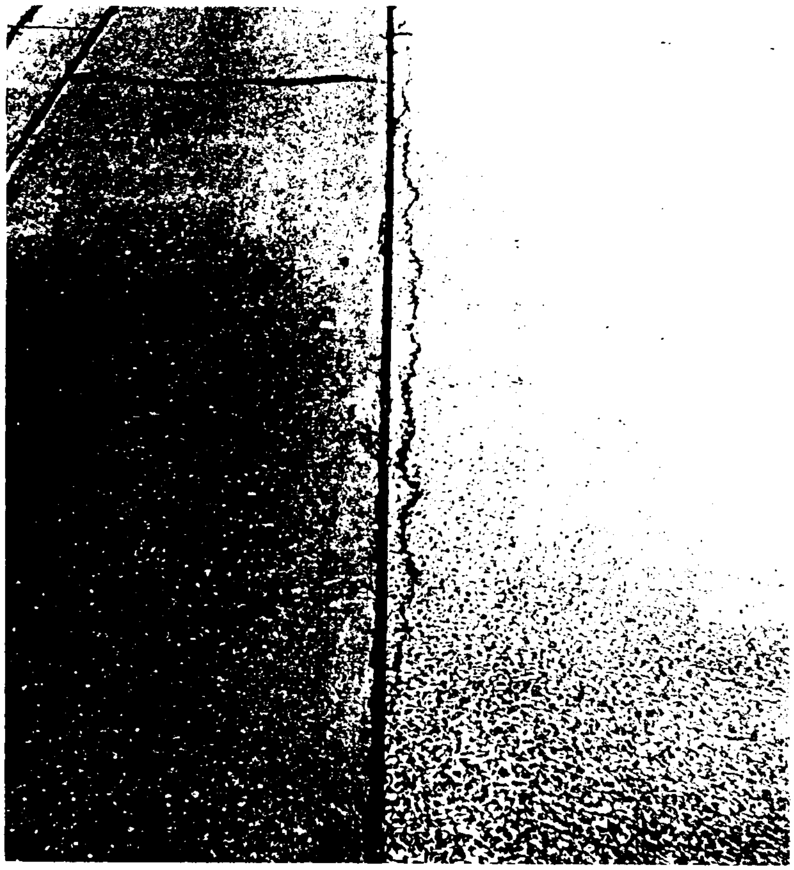

Figure 3.10: Cracking near RMP/PCC interface on Operations Apron

The Hanger 6 Apron appeared to be in the best condition of the three adjacent RMP aprons (Figure 3.11). Virtually no cracking or other deficiencies were noted on this apron. The superior condition of this apron as compared to the other two RMP aprons is not surprising considering the fact that this apron consisted of a full-depth reconstructed profile while the other two aprons involved overlaying over old JPCC.

Figure 3.11: Overall view of RMP-surfaced Hanger 6 Apron

Fort Campbell Army Airfield, Kentucky

An RMP aircraft warm-up apron located at the Fort Campbell Army Airfield, Kentucky was inspected in December 1995. This project site was constructed during September 1992, making the RMP about three years and three months old at the time of the inspection. The warm-up apron is located just off of the north end of the airfield's main runway and covers approximately 6,250 sq m in surface area. The pavement structure consists of an old JPCC pavement that is overlaid with 50- to 150-mm of AC and then 50-mm of RMP. The RMP apron has a 6-m-wide dense-graded AC shoulder around three sides with the fourth side being adjacent to a 38-cm-thick PCC taxiway. An overall view of the RMP warm-up apron is shown in Figure 3.12.

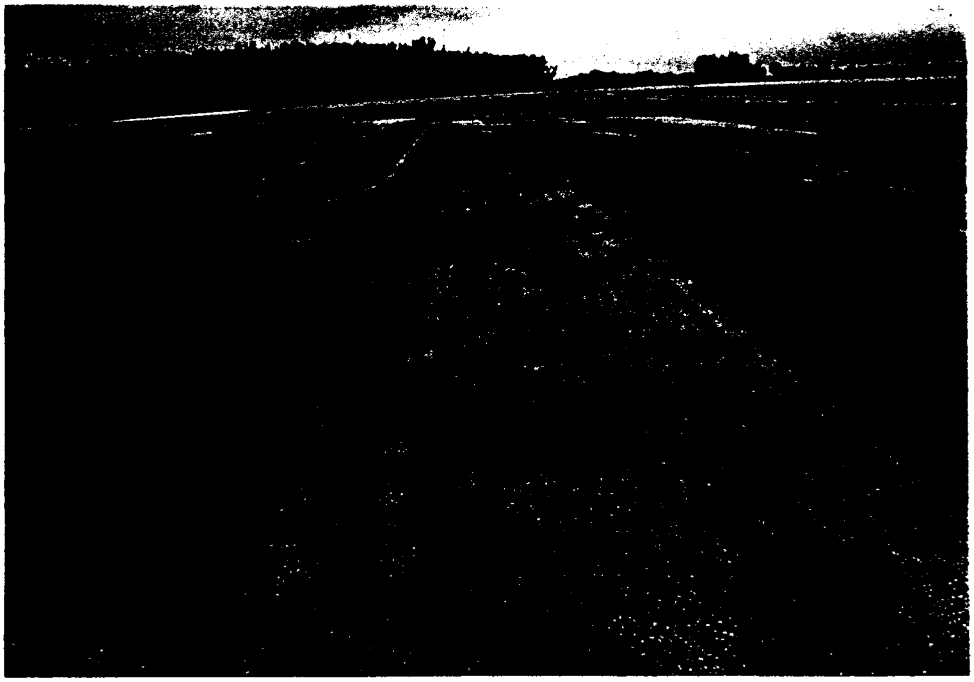

Figure 3.12: Overall view of RMP warm-up apron at Fort Campbell Army Airfield

The entire perimeter of the RMP apron was inspected to observe the condition of the joints between the RMP and the adjacent AC shoulder and PCC taxiway. The joint between the RMP and AC shoulder was in excellent condition, with no separation or spalling. The joint between the RMP and PCC taxiway had slight spalling of less than 15 mm deep along the top edges of both the RMP and PCC (Figure 3.13).

Figure 3.13: Spalling of material adjacent to PCC (left) and RMP (right) interface at Fort Campbell Army Airfield

The RMP apron at Fort Campbell had been closely inspected by Army personnel in 1993 and in 1994, and the December 1995 inspection was the first time that any reflective cracks were noticed. These cracks were presumed to be reflective since they ran for a considerable distance in a relatively straight direction and they were widely scattered throughout the RMP section in both the transverse and longitudinal directions (Figure 3.14). In a few instances, the underlying JPCC slab corners could be identified by connecting perpendicular cracks. There did not appear to be any spalling of RMP material adjacent to these cracks.

Local personnel at Fort Campbell reported a single incident involving the skid resistance of the RMP apron. In August 1993, approximately one year after construction, an A-10 aircraft had reportedly slid on the RMP apron while taxiing in a heavy rain storm. Isolated areas of over-grouted RMP were subsequently shot blasted and the problem reportedly had not reoccurred.

Figure 3.14: Reflective cracks in RMP apron at Fort Campbell Army Airfield

SITE INSPECTIONS AND SAMPLING OF NEW PROJECTS

During the course of this research study, the author had the opportunity to inspect two new RMP project sites as they were being constructed. This allowed for field samples to be taken during the routine quality control sampling for these projects, and these samples were later used to help establish the validity of at least some of the engineering properties measured on laboratory-produced samples. Also, documenting the conditions during construction at these new RMP projects would be valuable information for future field evaluations. These two RMP projects are described in the following sections.

Altus Air Force Base, Oklahoma

Two new airfield taxiways were constructed using RMP at Altus AFB, Oklahoma during June 1995. The RMP taxiway construction was part of a $28M airfield improvement project which included a new parallel runway, short field assault strip, and additional airfield lighting. RMP was selected for the new taxiways to provide better resistance to the damaging effects from high volumes of channelized cargo aircraft traffic. The total pavement area surfaced with the 50-mm-thick RMP material was approximately 10,500 sq m. Figure 3.15 shows a typical view of one of the two RMP taxiways at Altus AFB.

Figure 3.15: RMP taxiways at Altus AFB

Two factors made the RMP construction at Altus AFB somewhat unusual. First, ambient temperatures during construction were in the 30-35 deg C range, which kept the temperature of the open-graded asphalt concrete in the 40-50 deg C range during grouting operations. Ideally, the temperature of the open-graded asphalt concrete should be at or below 38 deg C to prevent rapid evaporation of the water in the grout during application. To combat this problem, very high water-cement ratios (0.75 compared to the typical 0.65-0.70 range) and the maximum allowable resin additive contents (3.5 %) were used. Secondly, much of the taxiway area had an unusually steep grade (approximately 4% compared to typical 1% maximum), which required extensive handwork to prevent the fluid grout from running downhill. High water and resin additive contents, as well as the necessary extra hand work, increased the chances for segregation of materials in the grout and likely reduced the ultimate strength of the final composite RMP layer.

Core samples recovered during routine quality assurance testing indicated full grout penetration in all areas of the two RMP taxiways (Figure 3.16). These core samples were retained for future laboratory testing in support of the research described in this report. The Altus AFB samples represented unusually harsh construction conditions and perhaps some of the poorest quality grout that is acceptable by current standards. Test results on these field samples will be discussed later in this report.

Figure 3.16: Core sample from RMP at Altus AFB showing complete penetration of grout into open-graded asphalt layer

McChord Air Force Base, Washington

In April 1996, RMP was used to reconstruct twelve aircraft refueling pads on an airfield parking apron at McChord AFB, Washington. Each refueling pad had approximately 900 sq m surfaced with a 40-mm thickness of RMP (Figure 3.17). Previous to the RMP reconstruction project, various fuel-resistant sealer materials had been used on the asphalt concrete parking apron to help resist damage from occasional fuel spills during routine aircraft refueling. Experience at this site proved that fuel-resistant sealers were not very effective in protecting the asphalt concrete pavement from fuel spill damage, and it was reported that approximately $20,000 had been spent every year on repair and maintenance work at this site. RMP was selected to resist fuel-damage and thereby eliminate the yearly repair and maintenance efforts.

During construction, the air void contents measured for the in-place open-graded asphalt concrete reportedly ranged from 30 to 35 percent and the grout viscosity averaged about 9.0 seconds. These are considered to be very typical values for field-constructed RMP materials. As part of the routine quality control testing program, one 100-mm-diameter core sample was cut from each of the twelve RMP parking spots (Figure 3.18). These cores were taken to visually determine if the grout had fully penetrated the open-graded asphalt concrete. All twelve core samples indicated full penetration of the grout into the 40-mm-thick open-graded asphalt concrete layer. The core holes were filled with low-shrinkage portland cement concrete material and all twelve cores were retained for future laboratory testing along with the field samples from Altus AFB. The tests that were conducted on these field core samples are described later in this report.

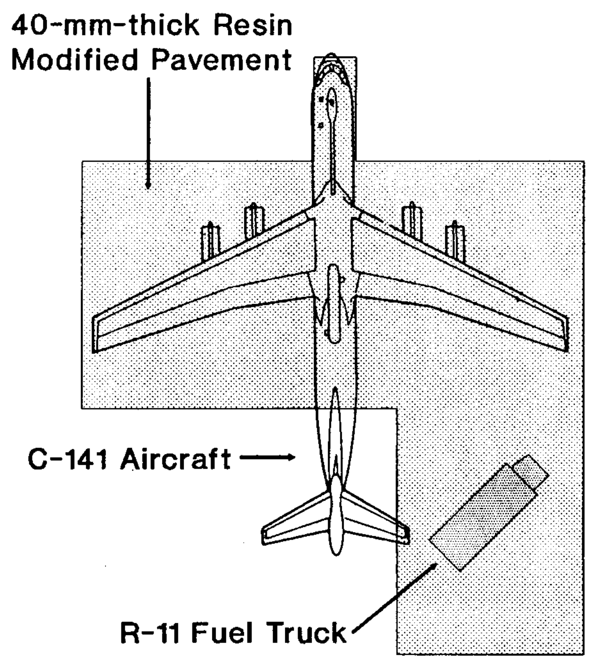

Figure 3.17: Geometry of RMP refueling pads on airfield apron at McChord AFB

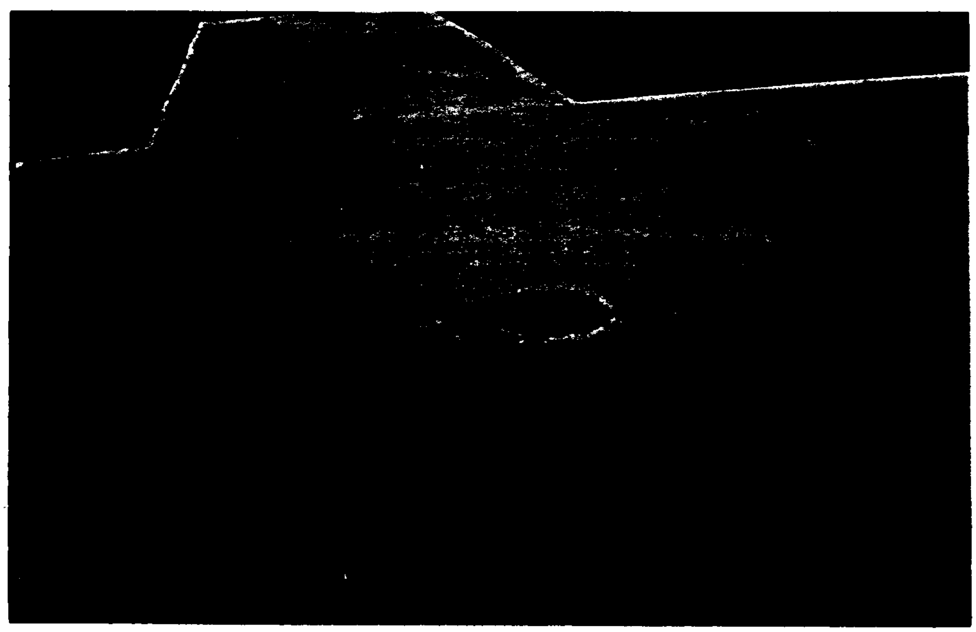

Figure 3.18: Core sample location from one of twelve RMP refueling pads

SUMMARY AND RESULTS OF FIELD EVALUATIONS

The field evaluations provided valuable insights into several critical field performance issues for RMP. The core samples gathered from the two recent RMP construction projects were also valuable in that they provided the opportunity to validate at least some of the engineering properties measured on the laboratory-produced RMP samples. A summary of the important performance-related issues discovered during the field evaluations is listed below. These issues formed the basis for the bulk of the laboratory test plan.

1. No evidence of wheel path rutting was found at any RMP location.
2. Reflective cracks from underlying concrete slabs were found at several locations. Cracks in the RMP surfacing were found in areas containing expansive clay materials in the subgrade. These failures indicate a need to characterize the stiffness and strength properties of the RMP material.
3. There was limited evidence of a potential for thermal cracking. There was evidence at two locations that RMP is thermally compatible with asphalt concrete, but not as thermally compatible with portland cement concrete. This indicated a need to determine the thermal properties of RMP, especially the coefficient of thermal expansion.
4. Although there was no physical evidence of fatigue cracking, this type of failure mode cannot be discounted as a potential problem with RMP simply because the sites evaluated were relatively young and the number of traffic applications were relatively low. Determining fatigue characteristics of pavement materials is a difficult challenge, but it was made a part of the laboratory evaluation to aid in the pavement modeling process.

5. The incident at Fort Campbell Army Airfield where the aircraft reportedly slid on a wet RMP surface pointed out the importance of determining the skid resistance of RMP. In fact, the lack of definitive skid resistance data has limited RMP usage to only low-speed traffic applications.
6. RMP thickness profiles continue to be designed based on a standard flexible pavement design with a replacement of the top 50 mm of asphalt concrete with 50 mm of RMP. The field evaluations indicate that this empirical approach has been sufficient in the short term, but there are not enough data on the RMP material itself to know if the current designs are overly-conservative, dangerously under-conservative, or somewhere in between the two extremes. This points out the need for a more rational mechanistic design approach, which is the ultimate goal of this research study.

Chapter 4: Laboratory Materials And Mix Designs

This chapter provides descriptions of the materials used to produce laboratory specimens of RMP for subsequent testing. The mix design procedures used to determine optimum blending formulas are also described. Finally, the methods used to physically produce and cure the test specimens are described at the end of the chapter.

MATERIALS

Each material component used to produce the RMP laboratory specimens is described below. References to the appropriate testing standards and specifications are provided in addition to the test results for each material.

Aggregates

The aggregates used in the open-graded asphalt concrete mixtures for all laboratory-produced RMP specimens consisted of crushed limestone from Alabama. These aggregates were separated into individual sieve sizes using a vibratory screening deck. The screening deck contained sufficient screens to separate the aggregates into the following sieve sizes: 12.5 mm, 9.5 mm, 4.75 mm, 2.36 mm, 1.18 mm, and 300 μm. The aggregate stockpiles produced by this process are listed in Table 4.1. The apparent specific gravity and water absorption values were measured for each stockpile according to the ASTM standard test methods C 127 and C 128 (ASTM, 1996a). These test results are also given in Table 4.1. Since the water absorption values for each aggregate stockpile were well below the maximum value allowed for standard definitions of non-absorptive aggregates (2.5 %), the existing mix design practice (Headquarters, Department of the Army 1997a) allowed for the use of apparent specific gravity values throughout the mix design process.

In addition to the sieve analysis, specific gravity, and percent water absorption measured for each aggregate stockpile, several additional tests were conducted to measure the relative quality of the coarse aggregates. Coarse aggregates are defined here as all aggregates retained by the 4.75 mm sieve. The test methods used and the desired test results were as specified by the existing guide specification for RMP (Headquarters, Department of the Army 1997b). These test results are given in Table 4.2.

Table 4.1: Open-Graded Asphalt Concrete Aggregate Stockpiles

Stockpile	12.5 mm	9.5 mm	4.75 mm	2.36 mm	1.18 mm	300 μm
Sieve Size	Percent Passing					
19.0 mm	100	100	100	100	100	100
12.5 mm	3.0	98.7	100	100	100	100
9.5 mm	0.6	11.7	99.5	100	100	100
4.75 mm	0.6	0.6	11.2	99.9	100	100
2.36 mm	0.6	0.6	1.0	17.3	98.8	100
1.18 mm	0.6	0.6	0.9	1.9	13.5	100
600 μm	0.6	0.6	0.9	1.7	2.7	99.2
300 μm	0.6	0.6	0.9	1.7	2.5	17.4
150 μm	0.5	0.6	0.9	1.7	2.5	4.7
75 μm	0.5	0.5	0.8	1.6	2.4	4.2
App. Spec. Gravity	2.73	2.73	2.74	2.72	2.69	2.77
% Water Absorption	0.2	0.3	0.5	0.1	0.6	0.2

Table 4.2: Coarse Aggregate Physical Properties

Test Method	ASTM Designation	Specification Requirement	Test Result
Los Angeles Abrasion	C131	≤40% loss	23.2%
Sodium Sulfate Soundness	C88	≤9% loss	2.7%
Flat or Elongated Particles	D4791	≤8%	0%

Asphalt Cement

The asphalt cement used in the production of open-graded asphalt concrete mixtures for all RMP specimens was a paving grade AC-20 produced by Lion Oil Company at an El Dorado, Arkansas refinery. The asphalt cement met the requirements prescribed by ASTM D3381 (ASTM 1996b), which is the industry standard for determining the suitability of viscosity-graded asphalt cements. The only requirements on the asphalt cement specified by the U.S. Army Corps of Engineers specification on RMP are that it meets the ASTM D3381 specification for an AC-10, AC-20, or AC-30 grade and that the original 25 deg C penetration (ASTM D5, ref. ASTM 1996b) is in the 40 to 100 range. The asphalt cement test results are given in Table 4.3.

Table 4.3: Asphalt Cement Test Results

Test Method	AC-20 Spec. (Table 2)	Test Result
Viscosity (60 C), Poise	2000 ± 400	2155
Viscosity (135 C), min. cSt	300	414
Penetration (25 C), 100g, 5 sec, min.	60	89
Flash Point, min., deg C	232	335
Solubility in trichloroethylene, %, min.	99.0	99.95
Tests on Residue from TFOT:		
Viscosity (60 C), max, Poise	10,000	4500
Ductility (25 C), 5 cm/min, min., cm	50	150+

Portland Cement

A single source of Type I portland cement, meeting the requirements of the ASTM C150 "Standard Specification for Portland Cement" (ASTM, 1996a) was used in the production of all laboratory-produced grout and RMP samples. The cement was produced by Blue Circle Cement Company at a Calera, Alabama production facility. Tests of the portland cement's chemical and physical properties were made, and the test results found in Table 4.4 indicate that this material did meet all ASTM C150 specification requirements for a Type I cement.

Table 4.4: Chemical and Physical Properties of Type I Portland Cement

Chemical Analysis	Test Result	ASTM C150 Requirements
SiO_2, %	20.3	--
Al_2O_3, %	4.6	--
Fe_2O_3,%	3.8	--
CaO, %	64.3	--
MgO, %	2.0	6.0 max
SO_3, %	3.0	3.5 max
Loss on ignition, %	1.5	3.0 max
Insoluble residue, %	0.13	0.75 max
Na_2O, %	0.08	--
K_2O, %	0.36	--
Alkalines - total as Na_2O, %	0.32	0.60 max
TiO_2, %	0.29	--
P_2O_5, %	--	--
C_3A, %	6	--
C_3S, %	61	--
C_2S, %	12	--
C_4, AF, %	12	--
Physical Tests		
Surface area, m^2/kg (air permeability)	399	280 min
Autoclave expansion, %	0.01	0.08 max
Initial set, min. (Gillmore)	170	60 min
Final set, min. (Gillmore)	285	600 max
Air content, %	6	12 max
Compressive strength, 3-day, MPa	25.4	12.4 min
Compressive strength, 7-day, MPa	32.9	19.3 min

Silica Sand

The silica sand used in the laboratory production of resin modified grout was known as an Ottowa foundry sand in the F-55 AFS grade classification. The sand had a rounded grain shape and a specific gravity of 2.65. The gradation of this silica sand was measured and is shown in Table 4.5.

Table 4.5: Silica Sand Gradation

Sieve Size	Percent Passing	
	Specification Requirement	Test Result
1.18 mm	100	100
600 μm	95 - 100	99
300 μm	--	73
150 μm	--	6
75 μm	0 - 2	1

Fly Ash

A non-hydraulic Class F fly ash was used as a mineral filler in the laboratory grout production. The fly ash was produced by Monex Resources at a Purvis, Mississippi coal burning electrical power plant. It was tested against the requirements of ASTM C618 "Standard Specification for Fly Ash or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete" (ASTM 1996a), and the results shown in Table 4.6 indicate that this material met all requirements prescribed by the ASTM standard.

Table 4.6: Chemical and Physical Properties of Class F Fly Ash		
Chemical Analysis	Test Result	ASTM C618 Requirements
SiO_2, %	54.8	--
Al_2O_3, %	30.2	--
Fe_2O_3, %	4.3	--
Sum, %	89.3	70.0 min
CaO, %	2.4	--
MgO, %	1.0	--
SO_3, %	0.3	5.0 max
Moisture content, %	0.1	3.0 max
Loss on ignition, %	4.4	6.0 max
Available alkalines (28-day), %	0.58	1.5 max
Physical Tests		
Fineness (45 micrometer), % retained	12	34 max
Fineness variation, %	0	5 max
Water requirement, %	100	105 max
Density, mg/m^3	2.31	--
Density variation, %	0	5 max
Autoclave expansion, %	-0.04	0.80 max
Strength activity index w/cement, 7-day, %	75	75 min
Strength activity index w/cement, 28-day, %	81	75 min

Resin Grout Modifier

The grout modifier used was the same material that has been used in all RMP applications in the United States. The modifier is a styrene-butadiene latex rubber resin produced by Jean Lefebvre Technology. The resin has a trade name of JLT Resin No. 407 and is commonly referred to as PL7. The physical data on this material, as reported by the manufacturer, are summarized in Table 4.7.

Table 4.7: Physical Properties of Resin Modifier	
Physical Property	Test Result
Specific Gravity	1 kg/l
Brookfield Viscosity (#1/10 rpm)	26 cps
Boiling Point	100 deg C
pH Level	9.5
Solubility in Water	miscible
Color	milky white
Percent Solids	47

MIX DESIGNS

The mix design tests and results for both the open-graded asphalt concrete and resin modified portland cement grout are described in the following sections. These tests are described in some detail since the RMP mix design procedure is not well known in the pavements industry. Reference is made to a newly standardized RMP mix design publication, which is found in the appendices of this report.

Open-Graded Asphalt Concrete

A mix design analysis was performed to determine the optimum blend of limestone aggregate stockpiles and optimum asphalt cement content. The optimum formula in this case renders a compacted open-graded asphalt concrete material that has an air void content very close to 30 percent and aggregates that are fully coated with asphalt cement. Air void contents from 25 to 35 percent are generally acceptable, but the normal variations found in field-produced asphalt concrete make a 30 percent air void target the most prudent choice when conducting laboratory mix designs. The procedures followed in both the open-graded asphalt concrete and the grout mix designs have been standardized and published by the U.S. Army Corps of Engineers (Headquarters, Department of the Army 1997a).

The first step in the open-graded asphalt concrete mix design was to determine the aggregate stockpile blending formula that produced a combined gradation closest to the center of the specified gradation band or tolerances. The optimum blending formula was found by a trial-and-error exercise to include various amounts of each aggregate stockpile previously described in Table 4.1. The optimum blending formula is shown in Table 4.8.

Table 4.8: Blending Formula for Open-Graded Asphalt Concrete Aggregates

Stockpile	12.5 mm	9.5 mm	4.75 mm	2.36 mm	1.18 mm	300 μm
Percentage by Weight	40%	10%	35%	5%	4%	6%

Sieve Size	Percent Passing	
	Spec. Limits	Optimum Blend
19.0 mm	100	100
12.5 mm	54 - 76	62.8
9.5 mm	38 - 60	51.5
4.75 mm	10 - 26	17.6
2.36 mm	8 - 16	11.1
1.18 mm	--	7.3
600 μm	4 - 10	6.6
300 μm	--	2.4
150 μm	--	1.2
75 μm	1 - 3	1.1

The apparent specific gravity of the combined aggregates representing the optimum blending formula was then calculated as follows (Asphalt Institute 1989):

$$G_{sb} = \frac{100}{P_1 / G_1 + P_2 / G_2 + \ldots + P_n / G_n}$$

where

G_{sb} = Apparent specific gravity of aggregate blend

$P_1, P_2, \ldots, P_n$ = Respective percentages of aggregate stockpiles 1, 2, etc.

$G_1, G_2, \ldots, G_n$ = Respective apparent specific gravity of aggregate stockpiles 1, 2, etc.

Substituting values from the predetermined blending formula renders:

$$G_{sb} = \frac{100}{\frac{40}{2.73} + \frac{10}{2.73} + \frac{35}{2.74} + \frac{5}{2.72} + \frac{4}{2.69} + \frac{6}{2.77}}$$

$$G_{sb} = 2.734$$

The next step in the open-graded asphalt concrete mix design was to estimate the optimum asphalt content. This was accomplished by using the following equation, which is based on previously determined aggregate properties (Roffee 1989b):

$$\text{Optimum Asphalt Content (OAC)} = 3.25\ (\alpha)\ \Sigma^{0.2}$$

where

$\alpha = 2.65/G_{sb}$

G_{sb} = apparent specific gravity of aggregate blend

Σ = conventional specific surface area = $0.21G + 5.4S + 7.2s + 135f$

G = percentage of material retained on 4.75-mm sieve

S = percentage of material passing 4.75-mm sieve and retained on 600 μm sieve

s = percentage of material passing 600-μm sieve and retained on 75-μm sieve

f = percentage of material passing 75-μm sieve

Substituting appropriate blending formula data renders:

$$\alpha = 2.65/G_{sb} = 2.65/2.734 = 0.969$$

$$\Sigma = 0.21G + 5.4S + 7.2s + 135f$$

$$= 0.21(0.824) + 5.4(0.110) + 7.2(0.055) + 135\,(0.011)$$

$$\Sigma = 2.648$$

$$OAC = 3.25\,(\alpha)\,\Sigma^{0.2}$$

$$= 3.25\,(0.969)\,(2.648)^{0.2}$$

$$OAC = 3.8$$

The estimated optimum asphalt content was used along with two asphalt contents above this value and two asphalt contents below this value to produce mix design samples in the laboratory. These five asphalt contents were evaluated at 0.2 percent increments: 3.4, 3.6, 3.8, 4.0, and 4.2 percent. Maximum theoretical specific gravities for open-graded asphalt concrete mixtures at each of these asphalt cement contents were calculated using the following formula (Asphalt Institute 1989):

$$G_{mm} = \frac{P_{mm}}{P_s / G_{sb} + P_b / G_b}$$

where

G_{mm} = maximum theoretical specific gravity of asphalt concrete mixture (no air voids)

P_{mm} = total loose mixture, percent by total weight of mixture = 100 percent

P_s = aggregate, percent by total weight of mixture

P_b = asphalt, percent by total weight of mixture

G_{sb} = apparent specific gravity of aggregate

G_b = specific gravity of asphalt

Substituting the appropriate aggregate and asphalt cement data, the maximum theoretical specific gravities for each mix design asphalt content were as follows:

G_{mm} @ 3.4% asphalt content (AC) = 2.589

G_{mm} @ 3.6% AC = 2.581

G_{mm} @ 3.8% AC = 2.573

G_{mm} @ 4.0% AC = 2.565

G_{mm} @ 4.2% AC = 2.557

The final step in the open-graded asphalt concrete mix design involved the production and evaluation of three 150-mm-diameter Marshall specimens at each of the five selected asphalt contents. An 1800-g batch of aggregates meeting the blending formula was prepared for each of the fifteen Marshall samples to be produced. The individual aggregate batches were dried and heated in a 145 deg C oven and the asphalt cement to be used in the specimen production was preheated to 135 deg C. The heated aggregates and proportionate amount of heated asphalt cement to create the proper asphalt content were combined and mixed in a mechanical mixer for approximately 15 to 30 seconds. This was a sufficient amount of time to thoroughly coat all aggregate particles with asphalt cement. The temperature of the asphalt mixture at the end of this brief mixing time was approximately 120 deg C. Immediately after mixing, the hot open-graded asphalt concrete mixture was placed in a 150-mm-diameter Marshall mold and compacted with 25 blows from a 4.5-kg Marshall hand hammer on one side of the specimen. The specimens were then air-cooled for a minimum of 4 hours before carefully removing them from the molds.

The air voids or voids total mix (VTM) for each compacted specimen was calculated based on its measured volume and dry weight. The equation used to calculate VTM is as follows:

$$\mathrm{VTM} = 100 - 100\,\frac{\mathrm{Wt}_{\mathrm{air}}}{(\mathrm{Volume})\,(G_{mm})}$$

where

$\mathrm{Wt}_{\mathrm{air}}$ = dry weight of specimen

Volume = $\pi/4\ D^2\ H$

D = specimen diameter

H = specimen height

G_{mm} = maximum theoretical specific gravity

The data resulting from this phase of the mix design analysis are given in Table 4.9.

Table 4.9: Open-Graded Asphalt Concrete Mix Design Results

Asphalt Content (%)	G_{mm}	Wt_{air} (g)	Specimen Diameter (cm)	Specimen Height (cm)	Vol (cm^3)	VTM (%)
3.4	2.589	1848.7	15.24	5.715	1042.7	31.5
3.4	2.589	1856.6	15.24	5.674	1035.3	30.7
3.4	2.589	1853.0	15.24	5.636	1028.3	30.4
						30.9 avg
3.6	2.581	1861.2	15.24	5.654	1031.6	30.1
3.6	2.581	1855.2	15.24	5.606	1022.8	29.7
3.6	2.581	1845.7	15.24	5.890	1074.7	33.5
						31.1 avg
3.8	2.565	1872.3	15.24	5.720	1043.6	30.1
3.8	2.565	1864.9	15.24	5.685	1037.1	29.9
3.8	2.565	1863.8	15.24	5.720	1043.6	30.4
						30.1 avg
4.0	2.573	1865.7	15.24	5.629	1026.9	29.4
4.0	2.573	1860.1	15.24	5.662	1033.0	30.0
4.0	2.573	1864.9	15.24	5.652	1031.1	29.7
						29.7 avg
4.2	2.557	1862.2	15.24	5.636	1028.3	29.2
4.2	2.557	1872.8	15.24	5.558	1014.0	27.8
4.2	2.557	1874.2	15.24	5.662	1033.0	29.0
						28.7 avg

Although the average VTM values for each of the five asphalt contents were relatively close to each other, the estimated optimum asphalt content of 3.8 percent did have the closest average VTM to the targeted 30 percent value. Therefore, 3.8 percent asphalt cement was used in the production of all further laboratory-produced open-graded asphalt concrete mixtures, which were in turn used to produce various types of RMP specimens. All laboratory-produced RMP specimens also had the same aggregate blending formula, mix temperatures, and compaction methods as previously described.

Resin Modified Portland Cement Grout

A mix design analysis was conducted to determine a suitable blend of the given grout materials for use in the production of laboratory samples. A suitable grout mix design is one that meets the batching percentage and viscosity requirements prescribed in the existing Corps of Engineers guide specification. The viscosity requirement is 8 to 10 seconds immediately after mixing when measured by the Marsh flow cone. The batching percentage tolerances for grout materials are given in Table 4.10.

Table 4.10: Required RMP Grout Mixture Proportions

Material	Percent by Weight
Type I Cement	34 - 40
Fly Ash	16 - 20
Sand	16 - 20
Water	22 - 26
Resin Modifier	2.5 - 3.5

The grout mix design tests began by mixing replicate samples of various formulations, all of which met the mixture proportion requirements specified in Table 4.10. Ten formulations were tested with two samples per formulation for a total of twenty grout viscosity tests. For each formulation, the batch weights were calculated based on 4000-g total sample weights. The dry ingredients (cement, sand, fly ash) for each sample were weighed and combined in 3.8 L cans that were subsequently sealed to prevent loss of material or contamination before mixing.

The grout samples were individually mixed in a 10-L mixing bowl, using a laboratory mixer with a wire whip mixing attachment. The dry ingredients were first placed in the mixing bowl, and then the bowl height was adjusted so that the wire whip was slightly above the bottom of the bowl. The mixer was turned on at a slow mixing rate and the appropriate amount of pre-weighed water was added. After all of the water was added, the mixer speed was increased until the grout was being thrown onto the sides of the mixing bowl. The grout was mixed at this high speed for 5 minutes, and then the appropriate amount of pre-weighed resin modifier was added to the grout. The grout was mixed at the high mixing speed for an additional 3 minutes before testing for Marsh flow viscosity.

Immediately after mixing the grout, the sample was poured from the mixing bowl into a smaller container for easier handling. Approximately 1100 mL of the grout was then poured into a Marsh flow cone (Figure 4.1). The flow cone was held with one hand and the cone outlet was plugged with a finger of the other hand. Immediately after the flow cone was filled to the 1100-mL fill line marked inside the cone, it was positioned over a 1000-mL glass or clear plastic graduated beaker. The cone opening was released and a stopwatch was activated simultaneously. The time of flow for 1 L of grout to exit the cone was then measured by visually observing the grout filling the beaker. This 1 L flow time was recorded as the Marsh flow viscosity. The results of the grout mix design tests are given in Table 4.11.

The grout mix design results indicated that three of the ten grout formulations (No's 5, 7, and 10) met the 8.0 - 10.0 sec. viscosity requirement. Blend 5 had the most optimum viscosity, however, and this grout formulation was selected and used for all following grout and RMP specimen production in this study.

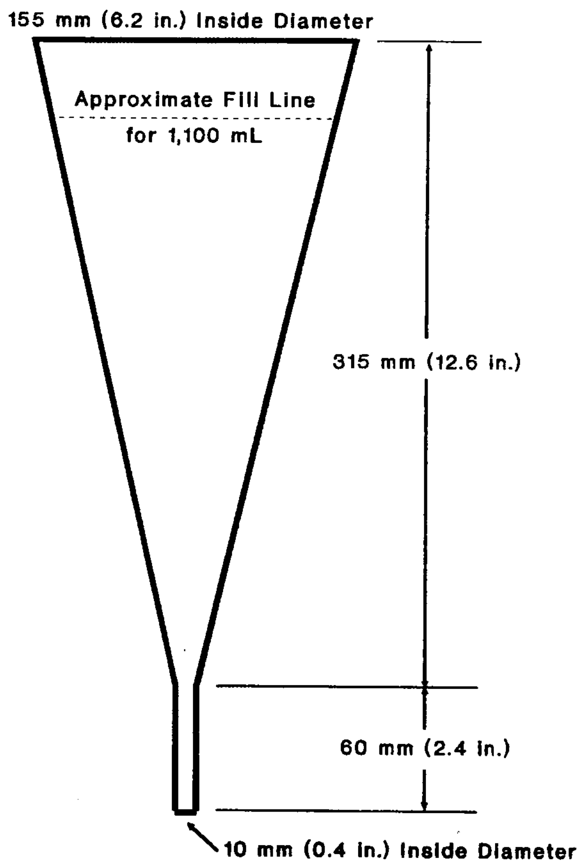

Figure 4.1: Dimensions of Marsh flow cone

Table 4.11: Resin Modified Grout Mix Design Results						
	Percent by Weight					
Blend No.	Portland Cement	Fly Ash	Sand	Water	PL7	Marsh Flow Viscosity (sec)
Spec.	34-40	16-20	16-20	22-26	2.5-3.5	8.0-10.0
1	36.5	19.0	19.0	22.0	3.5	17.5 / 16.5
						17.0 avg
2	37.0	18.4	18.4	23.0	3.2	15.0 / 14.5
						14.8 avg
3	37.0	18.0	18.0	24.0	3.0	14.5 / 13.5
						14.0 avg
4	36.6	17.6	17.6	24.8	3.4	11.5 / 12.0
						11.8 avg
5	36.6	17.1	17.1	25.7	3.5	9.0 / 9.0
						9.0 avg
6	36.0	19.2	16.0	25.3	3.5	10.0 / 11.0
						10.5 avg
7	36.0	16.0	19.2	25.3	3.5	10.0 / 10.0
						10.0 avg
8	34.0	19.4	19.3	23.8	3.5	11.5 / 11.5
						11.5 avg
9	33.8	19.3	19.2	23.7	4.0	11.0 / 10.0
						10.5 avg
10	34.7	17.9	17.9	26.0	3.5	9.0 / 9.5
						9.3 avg

LABORATORY SPECIMEN PRODUCTION

Laboratory specimens of RMP were generally produced in one of two forms, depending upon test method requirements: 100-mm-diameter by 50-mm-thick cylinders or beams of various sizes. The cylinders and beams were produced with the same materials and in the same manner so as to create uniformity between all laboratory specimens. Also, production and curing techniques were designed to simulate field conditions as much as possible.

The cylinders were produced by first compacting 150-mm-diameter by 50-mm-thick open-graded asphalt concrete specimens, using the same materials and procedures as previously described for the mix design phase of this study. After allowing the open-graded asphalt concrete material to air-cool overnight in the molds, the bottoms of the molds were sealed with duct tape in preparation for grouting. Batches of grout were mixed using the same materials and procedures as used before during mix designs, and each batch was tested for viscosity to ensure that all grout used for specimen production had a Marsh flow viscosity of between 8.0 and 10.0 seconds. After viscosity testing, each batch of grout was used to fill one or more open-graded asphalt concrete samples that were placed on a vibrating table. The vibrating table was used to simulate the vibratory roller used in the field construction of RMP. Grout was continually poured onto the top of each open-graded asphalt concrete specimen until the specimen was fully-saturated.

The freshly grouted specimens were allowed to air cure in the laboratory for a number of hours until the surface bleed water had evaporated. At this time, the surface of each RMP specimen was covered with a thin coating of a pavement curing compound with a brush. If possible, the specimens were allowed to remain in the molds for 28 days before testing. If it was necessary to remove the specimens from the molds, they were carefully ejected after a minimum of two days in the molds, and then immediately wrapped (sides and bottom) with cellophane cling wrap and duct tape for the remainder of the 28-day cure period.

After 28 days of curing and immediately before testing, all 150-mm-dia specimens were trimmed down to a 100-mm diameter required by most of the test methods that followed and to remove the irregular amounts of excess grout paste that naturally occurred during the molding process (Figure 4.2). This excess paste was thought to be non-representative of field-constructed RMP, and therefore needed to be removed.

Beams were produced similarly except for compaction of the open-graded asphalt mixture in the molds. Volumetric calculations were made for each mold size to determine the appropriate amount of hot mix required to render a 30 percent air void content in the final mixture. This exact amount of hot, open-graded asphalt concrete mixture was placed in each mold, and the mixture was compacted by applying a slow, vertical load on top of the specimen's surface (Figure 4.3). Air voids were measured on three beams for each beam size produced to validate the volumetric calculations. All beams produced had air void contents in the 29- to 31- percent range which indicated that this method of specimen production was acceptable. The beam samples were grouted (Figure 4.4) and cured in the same manner as previously described for cylindrical samples.

Figure 4.2: Core rig used to cut 100-mm-dia cores from 150-mm-dia specimens

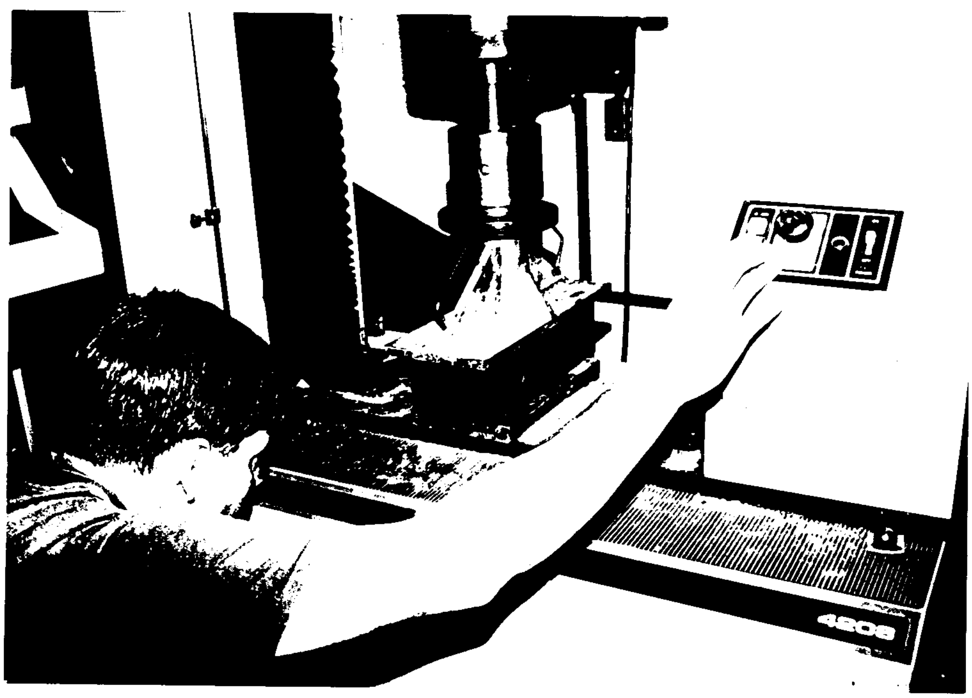

Figure 4.3: Compacting open-graded asphalt concrete beam sample

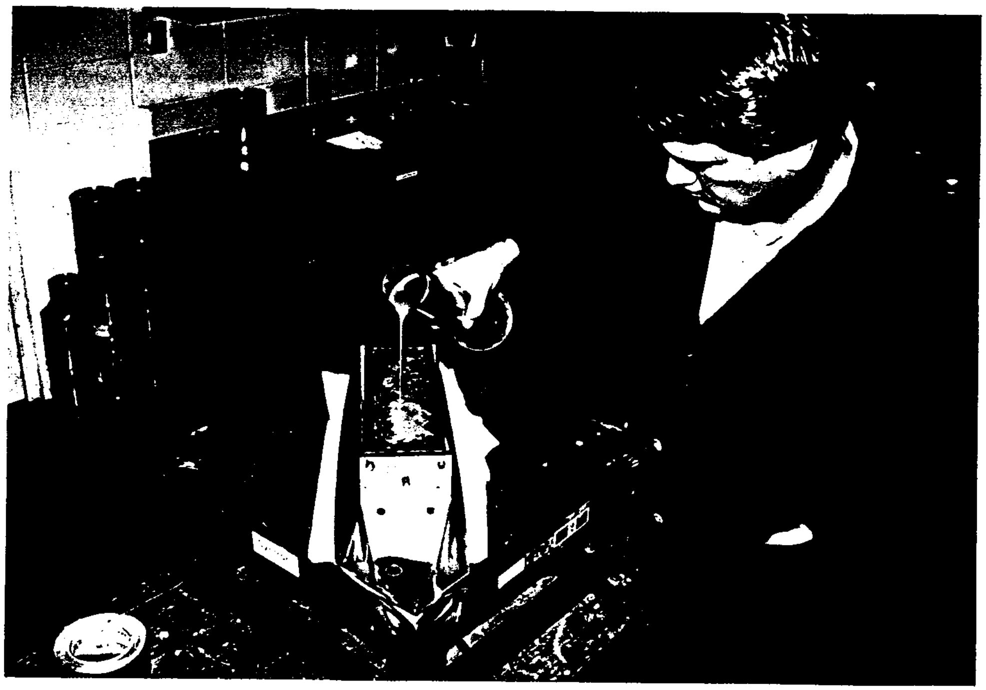

Figure 4.4: Applying grout to beam sample on vibrating table

Chapter 5: Strength Properties

Strength is the most common engineering property measured for pavement surfacing materials. The strength of these materials has a direct influence on the pavement's ability to resist traffic loads. In addition, pavement surfacing strength properties are usually easier to measure than other engineering properties, such as stiffness, and are therefore routinely used in correlations to predict these other properties.

Only a limited amount of strength data are available that describes RMP. Even though the pavement design approach proposed by this study focuses on measured stiffness and fatigue properties as design inputs, standard pavement strength properties need to be established for RMP for several reasons. First, strength measurements of RMP compared to the strength data of more traditional pavement surfacing materials (asphalt concrete and portland cement concrete) can provide a basic understanding of this material's physical nature. Secondly, it is quite possible that others referencing this study in the future may use strength properties as inputs for a different design methodology or analysis.

The following sections of this report describe the strength tests conducted on laboratory-produced and field samples of RMP, as well as the implications of the test results.

INDIRECT TENSILE STRENGTH

Indirect tensile tests were conducted on groups of laboratory-produced and field-recovered samples of RMP. These RMP data were compared to two sets of indirect tensile data on asphalt concrete samples made with the same limestone aggregate used in the RMP samples. The tests were conducted over a range of test temperatures to evaluate temperature sensitivity and to provide the required loading parameters for subsequent resilient modulus tests at these same temperatures.

The testing protocol used was ASTM D4123 “Indirect Tension Test for Resilient Modulus of Bituminous Paving Mixtures” (ASTM 1996b). Tensile strength was measured by placing a cylinder of RMP horizontally between two loading plates and loading the specimen across its diameter until failure (Figure 5.1). This loading configuration subjects the centerplane between the loading

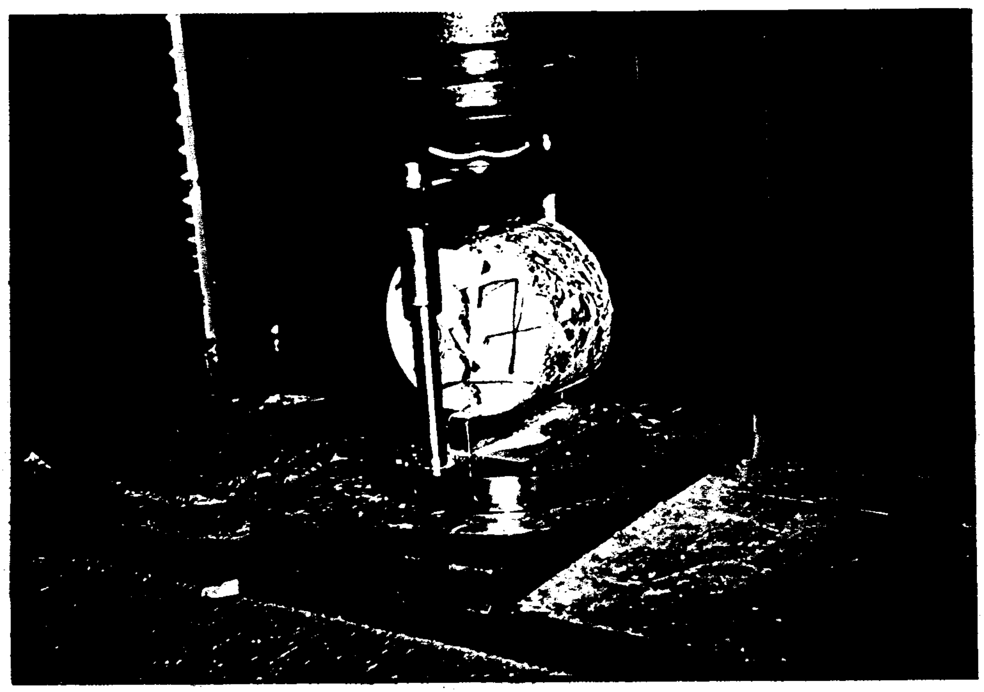

Figure 5.1: Indirect tensile strength testing of RMP sample

plates to a nearly uniform tensile stress, which results in specimen failure. The vertical load is applied to produce a constant deformation rate of 50 mm per minute until failure. The ultimate load is recorded at failure and is used to calculate tensile strength with the following equation:

$$TS = 2P/\pi\, t\, D$$

where:

TS = tensile strength, Pa

P = ultimate load required to fail specimen, N

t = thickness of specimen, m

D = diameter of specimen, m

Indirect tensile strength tests were conducted at three temperatures (5, 25, and 40 deg C) to simulate various pavement conditions. All laboratory-produced RMP samples were tested at 28 days after grouting. These laboratory samples were produced as 150-mm-dia. by 50-mm-thick cylindrical specimens that were left in the molds while air-curing in the laboratory. Just before testing, all specimens were cut into 100-mm-dia specimens using a portable pavement coring rig, as previously described in Chapter 4.

The field samples of RMP were recovered from two different sites. Nine 100-mm-dia samples were recovered from a June 1995 RMP airfield taxiway project at Altus AFB, Oklahoma. These samples were approximately 24 months old when tested. Another nine 100-mm-diameter samples were collected from an

April 1996 RMP airfield parking apron project at McChord AFB, Washington. The McChord samples were approximately 14 months old when tested.

The asphalt concrete indirect tensile strength data were produced by two previous research studies conducted at WES. The author conducted indirect tensile tests in 1987 on various asphalt concrete mixtures using a limestone aggregate from the same source as used in the study reported here (Anderton 1990). The data taken from this previous study represent the U.S. Army Corps of Engineers typical dense-graded, 19-mm maximum-sized aggregate, asphalt concrete mixture made with paving grade AC-20 asphalt cement. The other asphalt concrete tensile strength data used for comparison here were reported on by Ahlrich (Ahlrich 1996). The 1996 Ahlrich tests were made on asphalt concrete samples made with the same gradation of limestone aggregates from the same source, and made with an AC-20 grade asphalt cement very similar to the asphalt cement used in the author's 1987 tests.

All of the indirect tensile strength data for the laboratory RMP samples, the field RMP samples, and the previously-tested asphalt concrete samples are given in Table C.1 of Appendix C. These data are summarized in Table 5.1 and graphically displayed in Figure 5.2. A comparison of these data groups indicates a number of important factors:

1. RMP appears to have about the same indirect tensile strength as asphalt concrete at cold pavement temperatures, but at moderate-to-hot pavement temperatures, RMP has two to three times the strength of asphalt concrete.
2. Similar to typical dense-graded asphalt concrete mixtures, RMP looses tensile strength with increasing temperature. This provides evidence of it's visco-elastic material nature.
3. The RMP laboratory samples were fairly good indicators of field tensile strengths, especially at 25 and 40 deg C.

Table 5.1: Summary of Indirect Tensile Strength Data		
Material	Test Temperature (deg C)	Tensile Strength (kPa)
RMP (Lab)	5	2525
RMP (Lab)	25	1561
RMP (Lab)	40	571
RMP (Altus)	5	2097
RMP (Altus)	25	1760
RMP (Altus)	40	590
RMP (McChord)	5	2085
RMP (McChord)	25	1613
RMP (McChord)	40	816
AC (Anderton)	5	2387
AC (Anderton)	25	667
AC (Anderton)	40	265
AC (Ahlrich)	25	673
AC (Ahlrich)	40	247

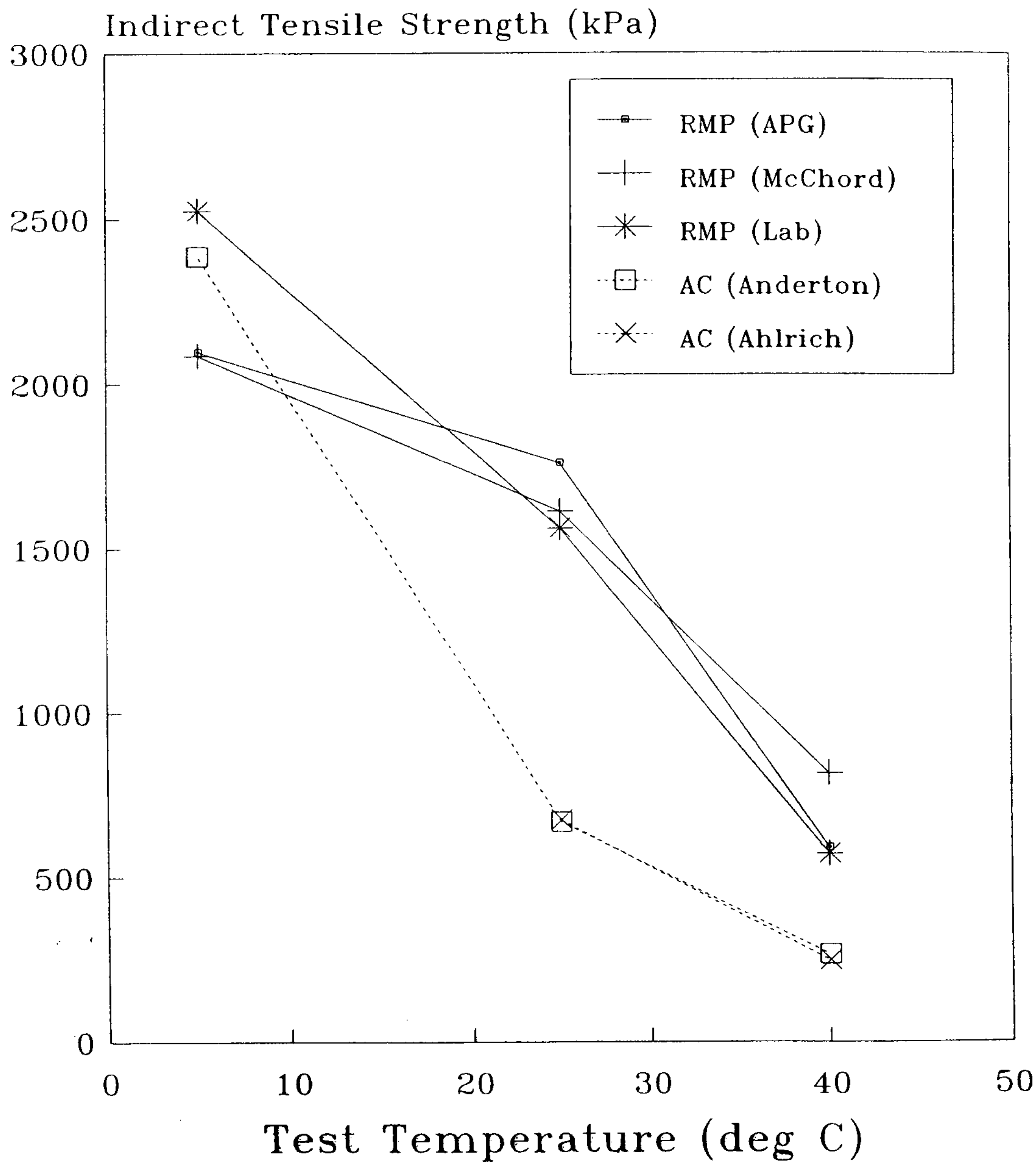

Figure 5.2: Indirect tensile strength versus temperature for RMP and AC samples

SPLITTING TENSILE STRENGTH

Splitting tensile strength tests were conducted on six RMP specimens in accordance with ASTM C496 "Splitting Tensile Strength of Cylindrical Concrete Specimens" (ASTM 1996a). The splitting tensile strength test is a standard test method for portland cement concrete materials, and is very similar to the indirect tensile strength test previously described. The only significant differences between these two test methods are the load rate and specimen size. Indirect tensile strength test loads are applied at a constant strain rate (50 mm per minute) while splitting tensile loads are applied at a constant stress rate (in the 689 to 1380 kPa per minute range). Indirect tensile specimens are generally 100-mm diameter by 50-mm thick while splitting tensile specimens require a thickness to diameter ratio of 0.94 to 2.10. The equation used to calculate splitting tensile strength is virtually identical to that for calculating indirect tensile strength, and is as follows:

$$T = 2P/\pi \, l \, d$$

where

- T = splitting tensile strength, Pa
- P = maximum applied load during testing, N
- l = length, m
- d = diameter, m

Six 150-mm-diameter by 100-mm-long RMP cylinders were produced in the laboratory using standard materials and procedures. After air curing in the molds for two days, the specimens were removed from the molds and then cored to produce 100-mm-dia by 100-mm long cylinders. These cylinders were then wrapped with cellophane cling wrap and duct tape on the sides and bottom. They were stored in the laboratory to complete the 28-day curing time before testing. The splitting tensile strength test results are given in Table 5.2.

Table 5.2: Splitting Tensile Strength Test Results

RMP Sample	Splitting Tensile Strength (kPa)	Statistics
1	837	
2	918	μ = 837 kPa
3	816	s = 57.7 kPA
4	756	v = s/μ = 6.9%
5	886	
6	811	

The general "quality" or repeatability of the splitting tensile strength test results can be estimated by the statistical variance (v). The American Concrete Institute considers statistical variances in the 10- to 20-percent range to be normal for strength measurements of portland cement concrete (ACI 1976). Therefore, the 6.9-percent variance for the RMP splitting tensile strength test results would indicate that these data are quite reliable relative to typical portland cement concrete tests.

The mean average (μ) splitting tensile strength, 837 kPa, can be compared to typical paving-quality portland cement concrete data for a relative strength assessment. Typical design values for flexural strength of paving quality PCC vary from 3450 to 5175 kPa (Departments of the Army and the Air Force 1987). The ratio of splitting tensile strength to flexural strength for normal PCC has been shown to be about 0.65 (Melis, et al 1985). This means that the splitting tensile strength of paving quality PCC is typically in the 2243 to 3364 kPa range. These data indicate that RMP generally has 65- to 75-percent less splitting tensile strength than PCC.

FLEXURAL STRENGTH

The flexural strength of six 75-mm by 75-mm by 275-mm RMP beams was measured according to the test method described by ASTM C78 "Flexural Strength of Concrete" (ASTM 1996a). The beams were produced and cured by the standard method previously described. During testing, each beam was supported on the stationary bottom plate at two points 225-mm apart, leaving a 25-mm overhang on each end. The top plate was used to apply the load at two points located equidistant from each other and from the bottom support points at 75 mm. These loading points are often referred to as third-points, thus giving this type of test the common "third-point loading" descriptor.

The load was applied at a constant strain rate of 2000 N/min, as prescribed by ASTM C78. At some point during the test, tensile cracking is initiated at the bottom of the beam specimen, and a successful test is one in which the initial crack is located within the central 75-mm-wide "loading zone" of the beam. All RMP tests met this criteria. A typical RMP flexural strength test is shown in Figure 5.3.

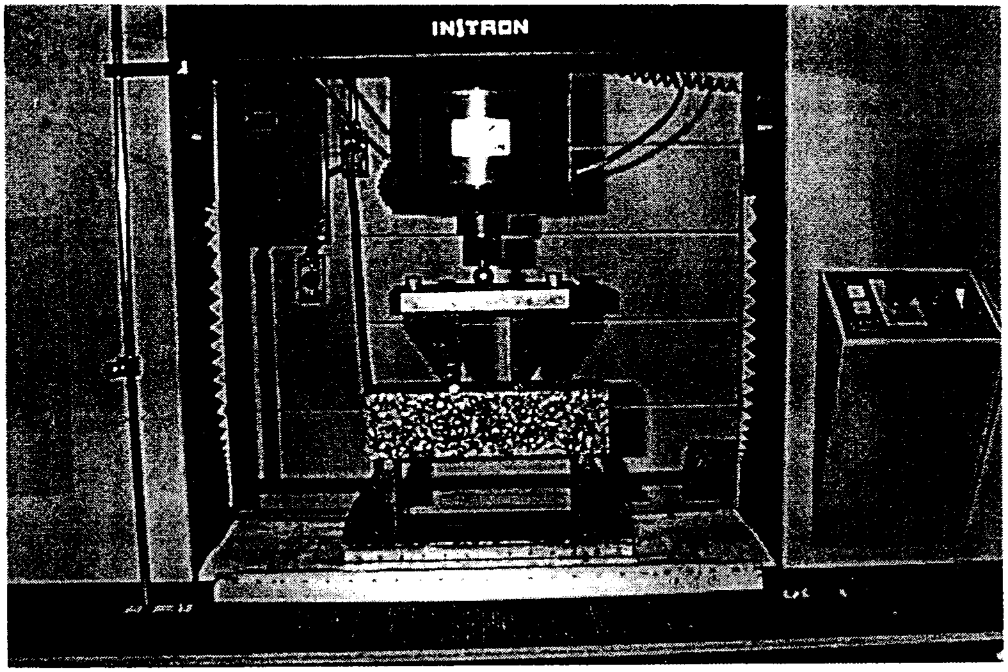

Figure 5.3: Third-point flexural strength test on RMP beam sample

The equation used to calculate flexural strength, commonly referred to as the modulus of rupture, is as follows:

$$R = PL/bd^2$$

where:

R = modulus of rupture, Pa

P = peak load at failure, N

L = span length, 0.225 m

b = beam width, 0.075 m

d = beam depth, 0.075m

The flexural strength test results are given in Table 5.3.

Table 5.3: Flexural Strength Test Results

RMP Sample	Modulus of Rupture (kPa)	Statistics
1	2214	
2	2079	μ = 2093 kPa
3	1989	s = 114.9 kPa
4	2023	$v = s/\mu$ = 5.5%
5	2256	
6	1999	

The statistical variance of this data is again compared to the ACI reference which considers statistical variances in the 10- to 20-percent range to be normal for concrete strength test results. The 5.5 percent value for this RMP data shows a high level of repeatability for the RMP data in comparison with typical portland cement concrete data.

As mentioned previously, typical design values for flexural strength of paving quality PCC vary from 3450 to 5175 kPa (Department of the Army and the Air Force 1987). By comparing the mean average flexural strength, 2093 kPa, to the typical PCC flexural strength range, it can be said that the RMP generally has 40- to 60- percent less flexural strength than PCC.

COMPRESSIVE STRENGTH

Resin Modified Pavement Material

Knowing the compressive strength of pavement surfacing materials has less practical purpose than knowing tensile or flexural strength values. This is true since these materials are generally much stronger in compression than in tension and the nature of their loading conditions will virtually always cause a tensile failure before a compressive failure. Nevertheless, compressive strength is the most common strength test conducted on rigid pavement materials since it is relatively easy to conduct when compared to other tests, and because there are numerous correlations existing which allows one to predict other material properties based upon the compressive strength (Carrasquillo 1994, Mindess and Young 1981). Compressive strength tests were conducted on RMP samples and grout samples to establish baseline values for comparisons with typical PCC values.

Compressive strength tests were conducted on six 100-mm-diameter by 100-mm-thick RMP specimens that were produced and cored in the same manner as previously described for the splitting tensile tests. The compressive strength tests were conducted in accordance with ASTM C39 "Compressive Strength of

Cylindrical Concrete Specimens" (ASTM 1996a). Each RMP specimen was capped with a sulfur capping compound according to the standard requirements prescribed in ASTM C39 and in ASTM C617 "Practice for Capping Cylindrical Concrete Specimens" (ASTM 1996a). The test was conducted by applying a compressive axial load to the capped RMP cylinders at a constant strain rate of 1.3 mm/min until compressive failure occurred. The compressive strength of the specimen was calculated by dividing the maximum load attained during the test by the cross-sectional area of the specimen. The compressive strength test results are given in Table 5.4.

Table 5.4: Compressive Strength Test Results

RMP Sample	Strength (kPa)	Statistics
1	3643	
2	4173	μ = 3869 kPa
3	3865	s = 196 kPa
4	4021	v = s/μ = 5.1%
5	3760	
6	3652	

The coefficient of variance for the RMP compressive strength data, 5.1 percent, indicates that these data are very consistent when compared to the normal 10- to 20-percent variance range cited by ACI for typical PCC strength data. The mean average compressive strength, 3869 kPa, may be compared to typical PCC compressive strengths. By applying the previously cited typical flexural strength range for paving-quality PCC (3450 to 5175 kPa) to the accepted ratio of PCC flexural strength to compressive strength (0.11 to 0.23, ref. Carrasquillo 1994), one can generalize typical paving-quality PCC compressive strengths to be within the 15,000 kPa to 47,000 kPa range. With this range of PCC data, it can be said that RMP has about 10 to 25 percent of the compressive strength that PCC has.

Grout Cubes

Compressive strength tests were conducted on numerous groups of resin modified grout cubes according to the testing standard ASTM C109 "Compressive Strength of Hydraulic Cement Mortars" (ASTM 1996a). This test method involves compression testing of 50-mm cubes using a constant strain rate of 2.5 mm/min until compressive failure occurs. The peak load at failure is recorded and then divided by the cross-sectional area to render the cube's compressive strength.

Two sets of cube compressive strength tests were conducted during this part of the study. First, tests were conducted on nearly-identical batches of grout cubes with the only difference being one group had the standard amount of resin additive and the other group had no additive. The grout formulation and materials were the same as those used throughout this study. These tests were conducted in an attempt to quantify possible strength gains or losses imparted by the additive. The second set of cube compressive tests involved testing identical batches of grout at different ages to track the expected strength gains with curing time. These data could possibly be used to establish curing time requirements on future RMP construction projects.

For the additive analysis, twelve cube samples with the resin additive and twelve cube samples without the additive were cast, air-cured in the laboratory for 28 days, then tested. The test results shown in Table 5.5 indicate the two data groups to be very similar. Even though the data of the unmodified grout samples were slightly more consistent and the mean average was about 10 percent higher than that of the modified grout samples, the differences are not significant enough to draw any important conclusions. The resin additive appears to have no appreciable effect on the grout's 28-day "ultimate" compressive strength.

Table 5.5: Test Results of Cube Compressive Strength Additive Analysis

Cube Compressive Strength (MPa)	
With PL7	No PL7
22.9	24.2
21.2	22.2
17.5	25.7
21.7	22.4
23.0	22.8
19.9	21.5
13.4	23.3
18.5	23.7
20.2	23.2
23.1	23.8
21.7	19.3
21.8	21.7
μ = 20.4 MPa	μ = 22.8 MPa
s = 2.8 MPa	s = 1.6 MPa
$v = s/\mu$ = 13.7%	$v = s/\mu$ = 7.0%

During the course of this research study, the author directed the laboratory mix design evaluations for three RMP field projects. The project sites were at Aberdeen Proving Ground (APG), Maryland, McChord Air Force Base, Washington, and Johnstown Army Reserve Center, Pennsylvania. Having access to the materials and optimum blends for these projects afforded the opportunity to test the curing time versus compressive strength relationships of different RMP grouts. Cube compressive strength tests were conducted on the optimum grout formulations for each of the three projects at 7, 14, and 28 days. The test results for these series of compressive strength tests are summarized in Table 5.6 and Figure 5.4. Test results of all replicate samples are given in Table C.2 of Appendix C. For comparative purposes, two sets of PCC cube compressive strength data are included in Figure 5.4. These data were taken from the British Code of Practice CP110 (Mindess and Young 1981), which describes cube compressive strengths of "standard" PCC materials at various ages.

Compressive strength gains with time appeared to be similar to that of standard PCC materials. As a general rule for PCC materials, the ratio of 28-day to 7-day strengths lies between 1.3 and 1.7 (Mindess and Young 1981). The 28-day to 7-day strength ratios of the APG, McChord, and Johnstown data are 1.8, 1.4, and 1.3, respectively. The McChord and Johnstown data show evidence of fairly low strength gains from 14 to 28 days. These data indicate that typical field applications of RMP may be ready for full traffic in about 21 days, especially when considering the strength-accelerating conditions of high ambient temperatures and wind found at many construction sites.

Table 5.6: Summary of Cube Compressive Strength Cure Time Analysis		
Project	Age (days)	Compressive Strength (MPa)
APG	7	12.8
APG	14	17.4
APG	28	22.8
McChord	7	15.5
McChord	14	19.3
McChord	28	21.3
Johnstown	7	17.1
Johnstown	14	20.4
Johnstown	28	22.1

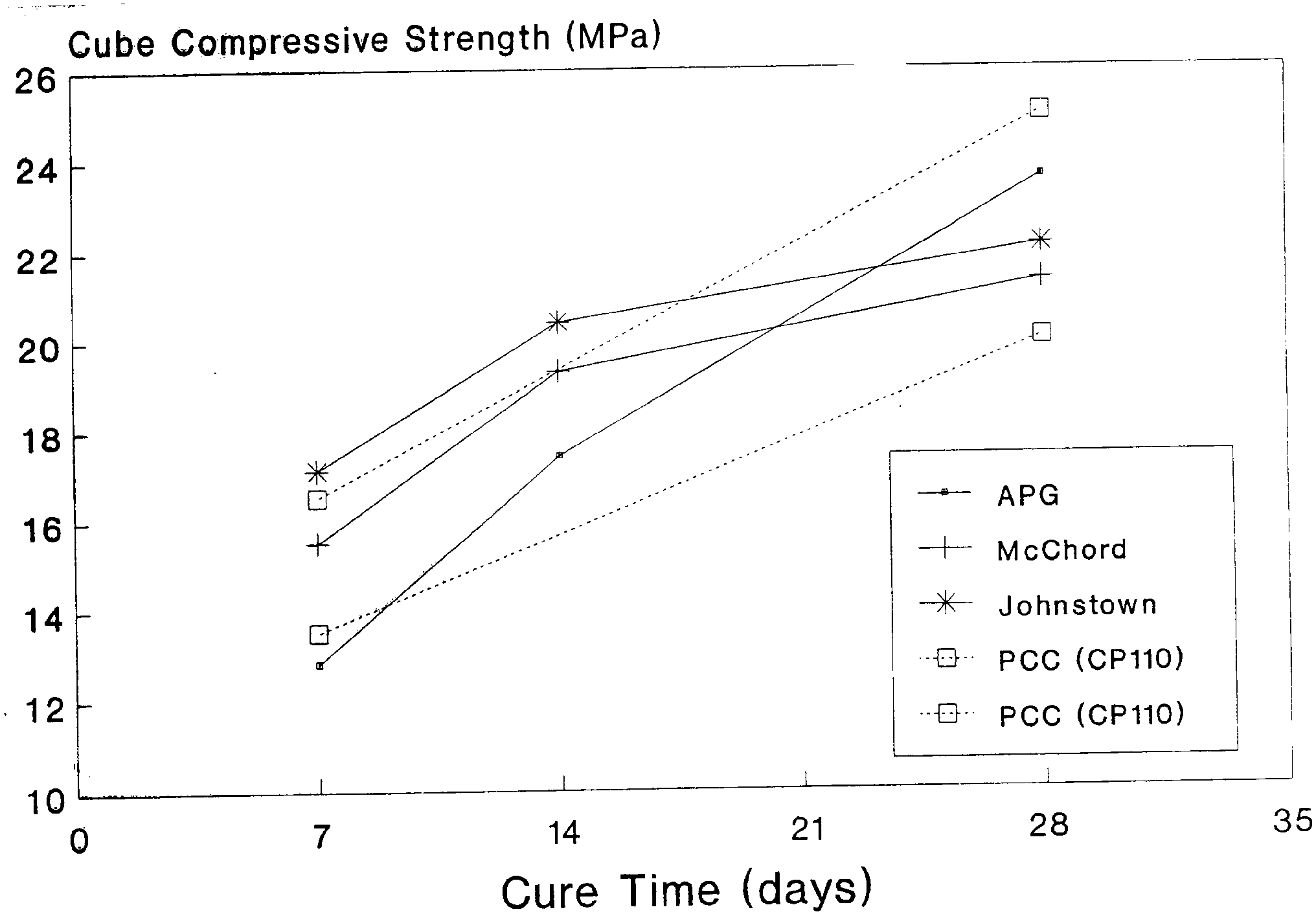

Figure 5.4: Cube compressive strengths versus age for various RMP and PCC grouts

Chapter 6: Elastic and Stiffness Properties

Two types of tests were conducted on RMP samples to help determine its elastic and stiffness properties: the Resilient Modulus by Indirect Tension Test and the Dynamic Fundamental Transverse Frequency Test. There are other tests available to determine modulus properties of pavement surfacing materials, but most of these tests require specimen sizes that prohibited them from being used for testing. For instance, most standard concrete stiffness tests require cylinders of 200- to 300-mm thickness. This thickness is not feasible for RMP samples, since the grout would not effectively penetrate this amount of open-graded asphalt concrete.

The elastic and stiffness properties were very important to this study since these data provide important inputs for most pavement design procedures. In the case of the linear elastic layered design procedure selected for analysis in this study, the two critical input properties are modulus and Poisson's ratio. The tests used to obtain these data, as well as their results, are described in the following sections of this chapter.

RESILIENT MODULUS BY INDIRECT TENSILE TEST

The resilient modulus is a dynamic test response defined as the ratio of the repeated diametric deviator stress to the recoverable diametric strain (Yoder and Witzak, 1975). The specimen size and set up are similar to that of the indirect tensile strength test previously described in this report, except that the load is designed to be non-destructive and dynamically applied. Also, electronic measuring devices are required to measure the relatively small stresses and strains incurred by the sample during testing.

There are two resilient modulus test standards used by the majority of pavement materials researchers in the United States: ASTM D4123 "Indirect Tensile Test for Resilient Modulus of Bituminous Mixtures" (ASTM 1996b) and AASHTO TP31 "Test Method for Determining the Resilient Modulus of Bituminous Mixtures by Indirect Tension" (AASHTO 1996). The test method used in this study follows the AASHTO procedures with modified equations from AASHTO TP9 "Test Method for Determining the Creep Compliance and Strength of Hot Mix Asphalt (HMA) Using the Indirect Tensile Test Device" (AASHTO 1996). The modified AASHTO test method was developed by researchers for the Strategic Highway Research Program (Roque and Buttlar 1992), and it features the use of subminiature electronic linear variable differential transducers (LVDT) mounted on the center of the specimen's flat face (Figure 6.1) rather than the traditional full-diameter, externally mounted deformation sensors. The advantage of the smaller strain gauge system is that the strain measurements are unaffected by the stress concentrations near the top and bottom loading platens, thus providing more reliable and consistent strain measurements. The applied load was a haversine pulse loading, with the load on for 0.1 sec and off for 0.9 sec.

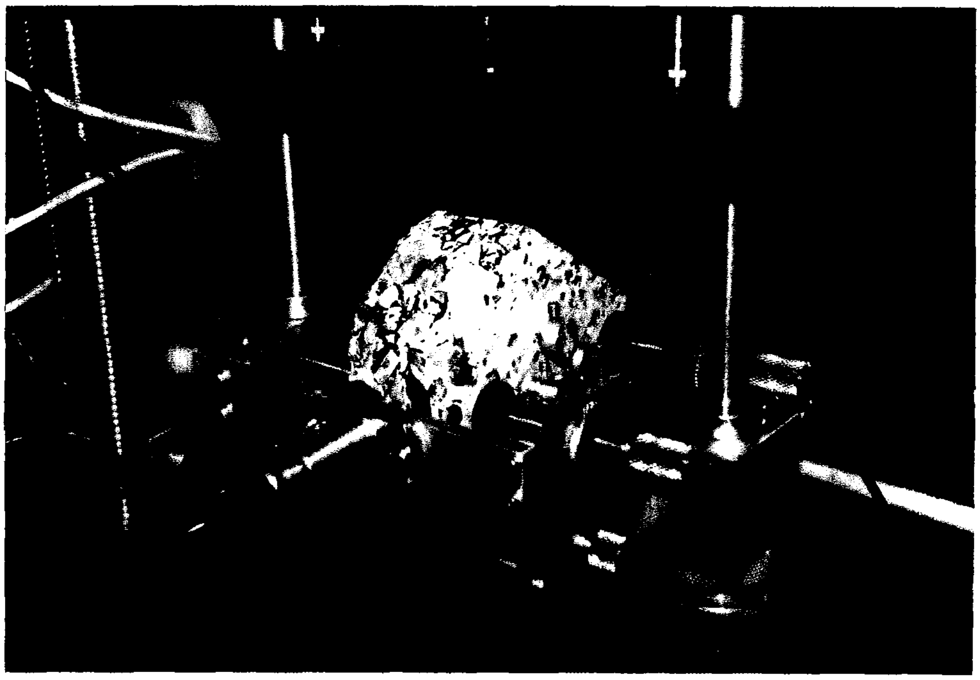

Figure 6.1: 25-mm LVDTs on RMP sample for resilient modulus testing

The equation used to calculate resilient modulus (E) for this test method is:

$$E = P\, G_L / H\, D\, t\, C_{compl}$$

where:

- P = applied load
- G_L = gauge length
- H = measured horizontal deformation
- D = specimen diameter

t = specimen thickness

C_{compl} = compliance correction factor

$= 0.6354\ (H/V)^{-1} - 0.332$

where V = measured vertical deformation

The equation used to calculate Poisson's ratio(ν) is:

$$v = -0.10 + 1.480(H/V)^2 - 0.778(t/D)^2(H/V)^2$$

where: H, V, t, and D are the same as described for the resilient modulus equation.

Resilient modulus tests were conducted on ten laboratory-produced RMP specimens and six field cores using three test temperatures. All test specimens were 100-mm in diameter and approximately 50-mm-thick. The laboratory samples were tested at 5 deg. C, then 25 deg. C, and finally at 40 deg. C. Three field cores from Altus AFB, Oklahoma and three field cores from McChord AFB, Washington were tested using the same temperature sequence. This testing program resulted in thirty modulus and Poisson's ratio data points for the laboratory samples and eighteen modulus and Poisson's ratio data points for the field cores. The complete test results are given in Tables C.3 and C.4 of Appendix C while the summarized test results are given here in Table 6.1. The Table 6.1 data are presented graphically in Figures 6.2 and 6.3.

Table 6.1: Summary of Resilient Modulus Test Results			
Specimen Origin	Test Temperature (C)	Resilient Modulus (GPa)	Poisson's Ratio
Lab	5	19.2	0.20
Lab	25	11.2	0.26
Lab	40	5.8	0.28
Altus	5	21.7	0.15
Altus	25	10.3	0.24
Altus	40	5.0	0.24
McChord	5	21.4	0.20
McChord	25	8.6	0.29
McChord	40	4.2	0.30

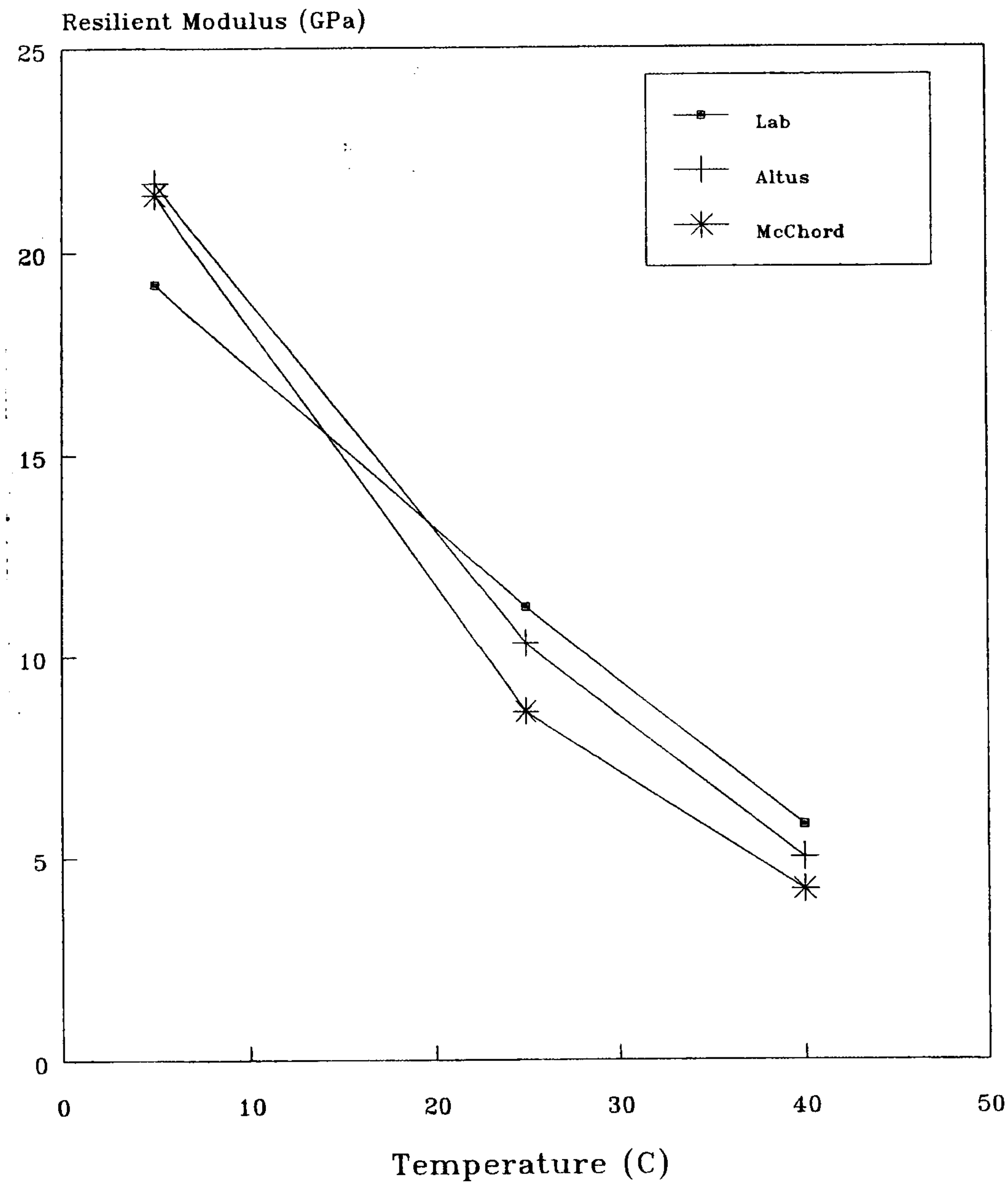

Figure 6.2: Effect of temperature on RMP resilient modulus

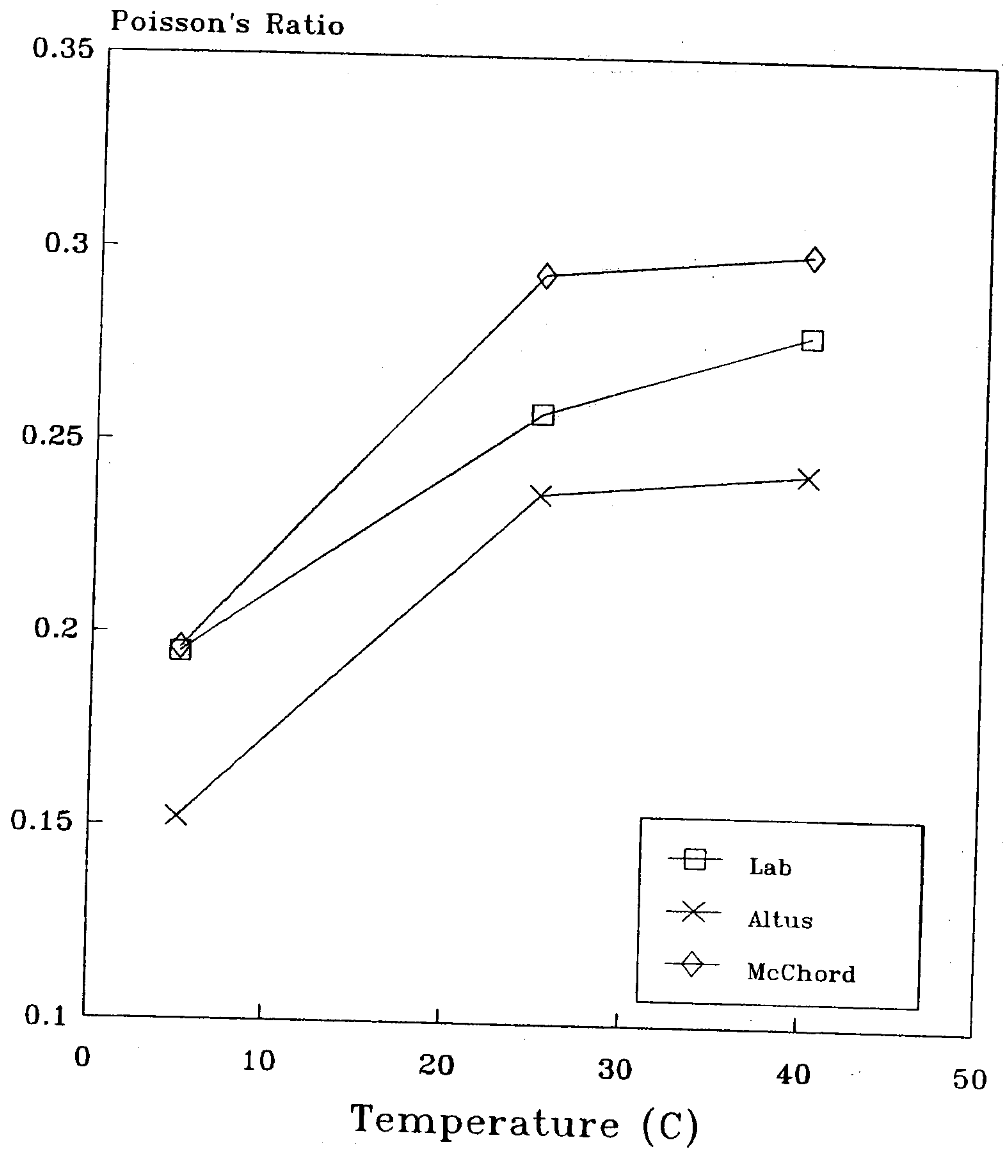

Figure 6.3: Effect of temperature on RMP Poisson's ratio

One of the most notable aspects of the resilient modulus test results is that the modulus is reduced and the Poisson's ratio is increased with increasing temperature. This provides further evidence of RMP's visco-elastic nature, allowing for direct comparisons to the stiffness characteristics of traditional asphalt concrete mixtures. Reported resilient modulus data for asphalt concrete mixtures have varied widely due to vast differences in aggregate types and gradations, as well as variable stiffness in the binder used in these mixes. In a study focused on additives for asphalt concrete mixtures (Anderton 1990), the author reported on resilient modulus tests conducted on fifteen different AC mixtures made with limestone aggregates similar to those used in this study. Resilient modulus values at 5 deg. C generally ranged from 11.2- to 23.7-GPa, from 2.3- to 13.3-GPa at 25 deg. C, and from 0.67- to 2.8-GPa at 40 deg. C. For comparison, Figure 6.4 displays these ranges with the RMP modulus ranges of Table 6.1. By observing these comparative modulus ranges, it can be said that the RMP behaves like a very stiff asphalt concrete mixture in low to moderate pavement temperatures; but at high pavement temperatures, RMP generally has two to seven times the stiffness of asphalt concrete.

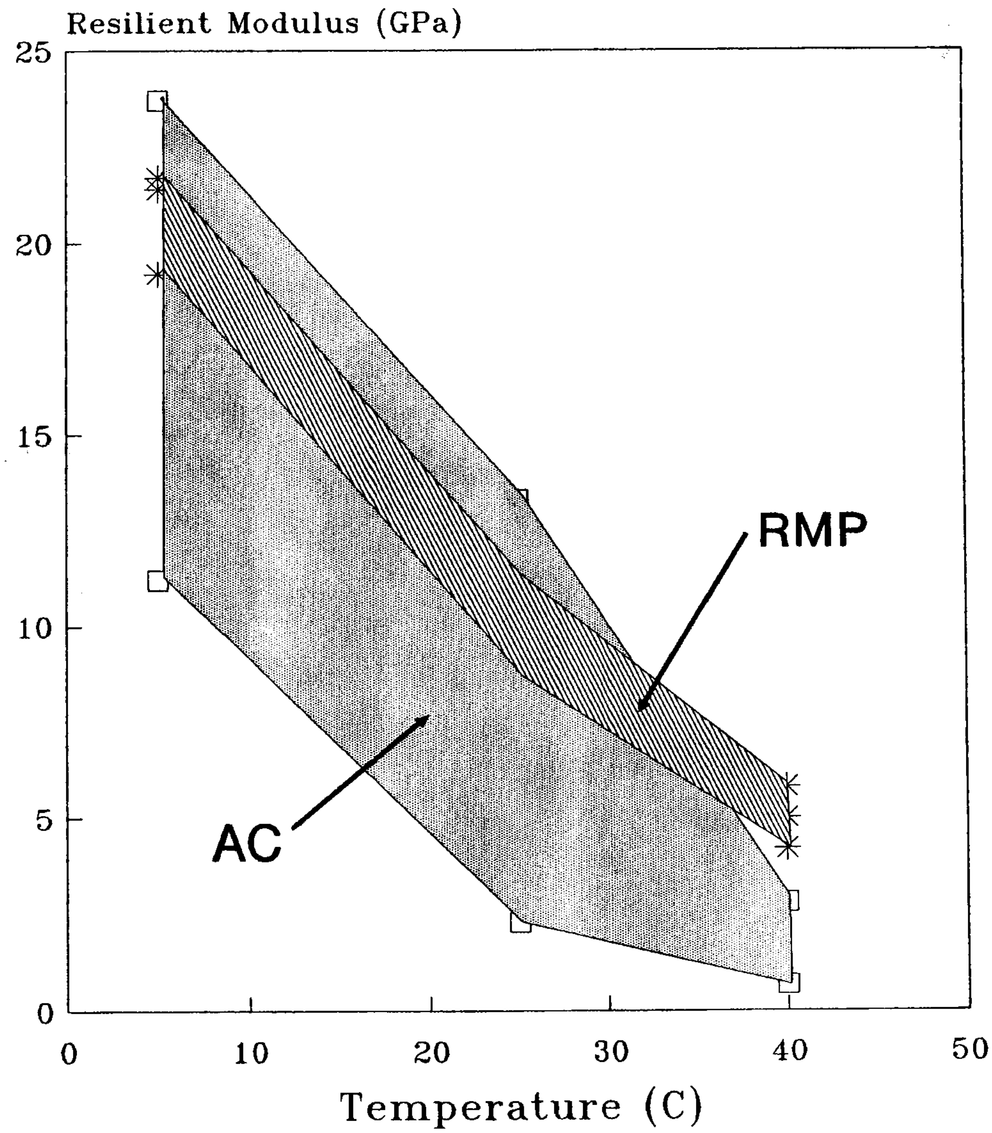

Figure 6.4: Typical resilient modulus versus temperature ranges for RMP and asphalt concrete (AC)

The range of Poisson's ratios for RMP displayed in Figure 6.2 is not surprising considering that the documented Poisson's ratio for portland cement concrete generally falls in the 0.15 to 0.20 range (Mindess and Young 1981), and for asphalt concrete the range is 0.25 to 0.40 (Roberts, et al 1991). When the Poisson's ratio is used in pavement design, most agencies propose the use of a single value, disregarding the limited effects of material and environmental variations. The Poisson's ratio value most often used for portland cement concrete in pavement design is 0.20, and for asphalt concrete the value used is usually 0.35. Selecting a single, representative Poisson's ratio from the high-temperature end of a particular range follows a conservative approach since most pavement damage generally occurs during moderate to warmer temperatures when pavement subgrades have thawed and asphalt concrete is less stiff and more vulnerable to deformation distresses. Following this logic, if a particular pavement design approach required a single Poisson's ratio value for RMP, then that value should be the average of all data at these temperatures, or 0.27.

DYNAMIC YOUNG'S MODULUS BY FUNDAMENTAL TRANSVERSE FREQUENCY TEST

The dynamic Young's modulus is a common measure of portland cement concrete stiffness. The test method described by ASTM C215 "Fundamental Transverse, Longitudinal, and Torsional Frequencies of Concrete Specimens" (ASTM 1996a) was used to measure the modulus of three 80-mm high, 112-mm wide, and 375-mm long RMP beams. The test generally involves simply supporting the beam specimens at two nodal points underneath the beam (at 0.224 of the length from each end of the specimen), and then forcing the beam to vibrate at various frequencies. These vibrations were induced by striking the beam at the midpoint of it's upper surface with a small metal hammer. The vibrations were picked up by an electronic transducer placed at one end of the upper specimen surface. The vibration frequencies were observed and captured on an oscilloscope, then the frequency giving maximum output was recorded as the fundamental frequency. Figure 6.5 depicts a typical test setup.

The dynamic modulus of elasticity (E) was calculated from the following equation:

$$E = C\,W\,n^2$$

where:

C = a constant derived from specimen dimensions and shape

W = weight of the specimen (kg)

n = fundamental frequency of vibration (Hz)

The dynamic modulus test results are given in Table 6.2. The dynamic modulus values derived from this test are in the same range as the resilient modulus values previously described for RMP. There is virtually no experience with comparing the results of these two modulus tests since one method is used exclusively for concrete materials while the other is typically only used for soils and AC materials. It appears that these tests are comparable for RMP from the test results of this study, and this agreement between test results serves to validate their use in pavement design procedures.

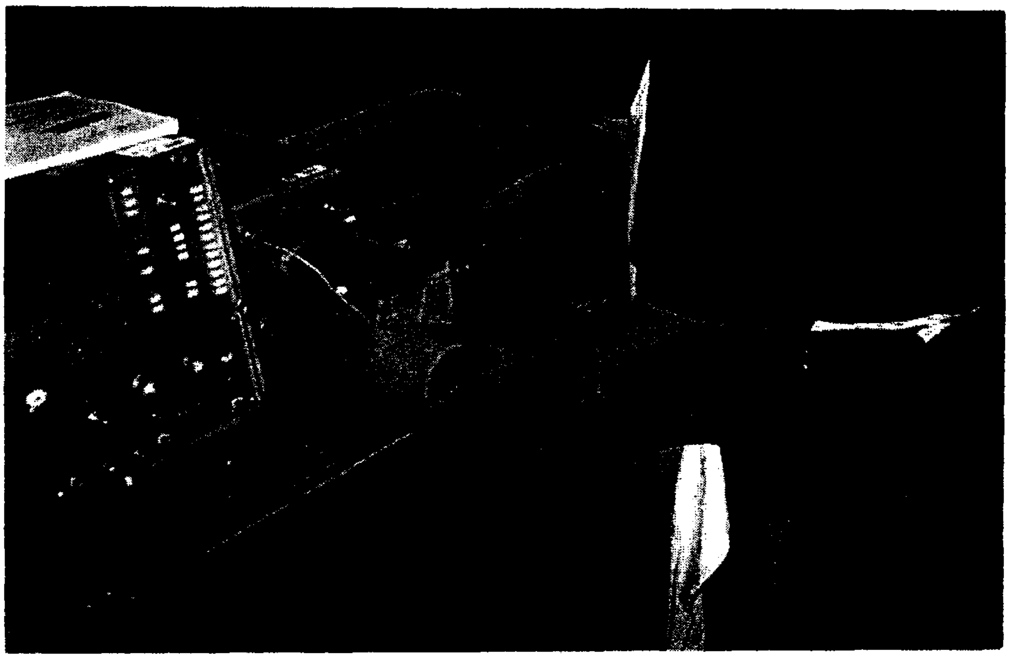

Figure 6.5: Typical setup for RMP fundamental frequency test

Table 6.2: Dynamic Modulus by Transverse Frequency Test Results				
Sample	Dimensional Constant - C	Weight (kg)	Fundamental Frequency (Hz)	Dynamic Modulus (MPa)
1	6.8×10^{-4}	7.72	1500	11,818
2	6.8×10^{-4}	7.60	1450	10,868
3	6.8×10^{-4}	7.75	1425	10,701

Chapter 7: Thermal Properties

Thermal properties of any pavement surfacing material can have a significant impact on the pavement's performance for a number of reasons. Pavement surfacings tend to expand at high temperatures and contract at low temperatures relative to the thermal coefficients of the large aggregates which represent the largest portion of the material by volume. These expansions and contractions are on a very small scale, relatively speaking, but when these materials are bonded to another material whose geometric constraints are forcing it to move in a different direction or at a different rate, then pavement cracks or buckling at pavement joints can result.

Besides the problems associated with expansions and contractions, pavement surfacings can deteriorate during freeze-thaw cycles when water is present. Pavements can also suffer accelerated deterioration during freeze-thaw periods when de-icing salts or other chemicals are present. The common failure mode in these conditions is commonly referred to as scaling.

These important thermal properties relating to pavement performance were unknown for RMP before this study was conducted. There was some evidence of a difference in thermal coefficients between RMP and PCC at two of the field sites inspected during the beginning of this study. Scaling resistance for RMP was unknown because of the lack of older sites in the United States. Two sets of tests were conducted on laboratory-produced RMP samples to evaluate these thermal properties in hopes that some important design principals could be developed to address these issues. These tests and their results are discussed in the following sections of this chapter.

COEFFICIENT OF THERMAL EXPANSION

The coefficient of thermal expansion was measured for two sets of RMP samples: one made with crushed limestone and one made with crushed siliceous gravel. These two aggregate types are known to have considerably different thermal coefficients, with limestone having the lowest value for common paving aggregates (6 X 10^{-6}/°C) and gravel having a relatively high value (11 X 10^{-6}/°C) (Mindess and Young 1981).

Concrete made with limestone has a thermal coefficient of about 8 X 10^{-6}/°C, while concrete made with gravel has a thermal coefficient of about 12.0 X 10^{-6}/°C (Mindess and Young 1981). Thermal coefficients of AC with different aggregate types is less of an issue because the binder tends to allow for stress relaxation, which dampens the effects of subtle variations between aggregate types. Because thermal coefficients are considered to be less critical in predicting performance for AC, the research and published data for these materials are very scarce. However, one collective study performed for the Strategic Highway Research Program (Janoo, et al 1995) reported the thermal coefficients of various AC materials measured in seven different research studies to be in the 17- to 30 X 10^{-6}/°C range.

Three RMP beam samples were fabricated for each of the two aggregate types. Aside from the aggregate type, all beams had the same aggregate gradation, asphalt content, and grout. The beams were 75 mm high, 75 mm wide, and 245 mm long. Each RMP beam was tested for thermal coefficient according to the standard procedure described in CRD-C 39-81 "Test Method for Coefficient of Linear Thermal Expansion of Concrete" (U.S. Army Corps of Engineers 1993). This test method generally involved measuring the length of each beam at various temperatures while in a water-saturated condition. A length comparator and metal gage studs cast in the ends of each beam were used to measure length changes (Figure 7.1). Length measurements were made with the following temperature conditioning sequence: 25°C water bath, 60°C water bath, 25°C water bath, 5°C water bath. The first 25°C water bath was used to allow the beams to undergo possible expansion due to saturation, while the second 25°C conditioning period

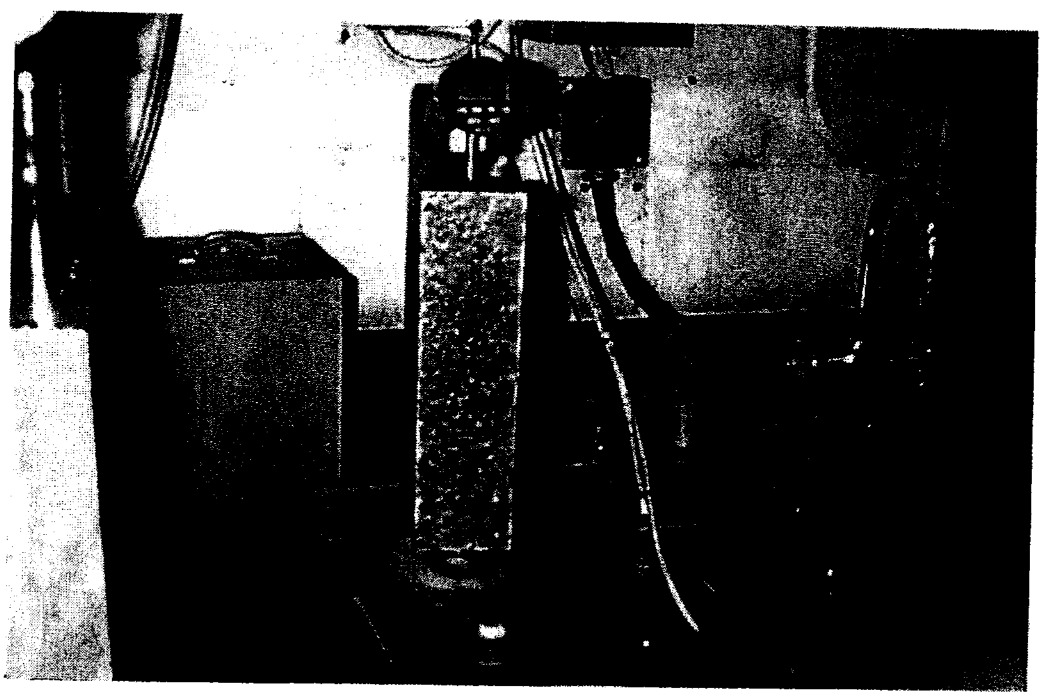

Figure 7.1: Measuring length of RMP beam sample for thermal coefficient test

was used to allow the beams to slowly contract without damage from thermal shock. Beam specimens were allowed to condition in each water bath until expansion or contraction was completed. Daily measurements were made to determine when this equilibrium had occurred. Equilibrium for each temperature conditioning sequence usually occurred in 24 to 48 hours.

The sample length measurements recorded at equilibrium for the 60°C and 5°C conditions were used in the following equation to calculate the coefficients of linear thermal expansion (C):

$$C = (R_h - R_c) / G \ \Delta T$$

where:

R_h = length reading at higher temperature (mm)

R_c = length reading at lower temperature (mm)

G = gage length between beam inserts (254 mm)

ΔT = difference in temperature of specimen between the two length readings (55°C)

The test results for the six RMP beam samples are given in Table 7.1. Comparisons between these test results and the thermal coefficients cited in the literature for PCC and AC lead to the following conclusions:

1. RMP has thermal coefficients in the same general range as PCC, which is about two to three times lower than that of HMA.

Table 7.1: Coefficient of Thermal Expansion Test Results			
Specimen	Aggregate	R_h-R_c (mm)	Thermal Coefficient (X 10^{-6}/°C)
1	Gravel	0.1625	11.8
2	Gravel	0.1625	11.8
3	Gravel	0.1725	12.3
			12.0 avg
4	Limestone	0.1500	10.7
5	Limestone	0.1525	10.9
6	Limestone	0.1525	10.9
			10.9 avg

2. RMP thermal coefficients appear to be less sensitive to aggregate type when compared to PCC, possibly owing to a damping effect caused by the asphalt cement surrounding and between each aggregate.
3. The deterioration near RMP/PCC pavement interfaces which was noted at the Fort Campbell Army Airfield and the Pope Air Force Base site inspections was not attributable to differences between the two thermal coefficients, as was speculated at first. The joint deterioration and cracking parallel to the joint was likely caused by the two pavement materials attempting to contract away from each other during periods of cold temperatures while the interface was mistakenly bonded.

FREEZING AND THAWING RESISTANCE

There are three main mechanisms by which portland cement-based pavement surfacings are damaged during freezing and thawing periods. If the permeability of the material is high enough, moisture can enter the internal pore spaces of the paste. When this moisture freezes, the ice crystals generate hydraulic pressures in the pore spaces as volumetric expansion occurs, and this leads to internal tensile stresses and cracking. It is also possible for certain aggregates to suffer similar damage when they absorb water and then undergo freezing conditions. Fractured aggregates near the surface may pop out, or when located near joints or at the bottom of the pavement surfacing where moisture is usually most available, gradual deterioration in the form of D-cracking or raveling may occur. Finally, scaling at the pavement surface can occur in the presence of deicing salts or chemicals even when the pavement surfacing is relatively impermeable to water intrusion. Freezing and thawing of moisture at the surface in these conditions can cause a gradual loss of small paste particles, resulting in a roughened and pitted surface.

The permeability of RMP has been established previously by researchers at Virginia Polytechnic Institute under contract to the Strategic Highway Research Program (SHRP) (Al-Qadi 1993 and 1994). As mentioned in Chapter 2 of this report, the focus of the SHRP research was to evaluate the RMP's suitability as a bridge deck material. Since moisture intrusion is a critical issue for bridge decks, the permeability of RMP was measured and compared to standard PCC materials.

The researchers concluded that RMP was two to three times more resistant to moisture and chloride intrusion than PCC.

Since permeability was predetermined to be very low for RMP, scaling resistance became the primary freezing and thawing property which needed to be determined. Tests on three laboratory-produced RMP samples were conducted according to the ASTM C672 test method "Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals" (ASTM 1996a). This test method generally involved ponding about a 6-mm-deep solution of calcium chloride and water on the surface of the RMP beam samples and then subjecting them to daily freezing and thawing cycles. The samples were placed in a -20°C freezer for 16 to 18 hours and then allowed to thaw in laboratory air (approximately 23°C) for 6 to 8 hours. After five of these daily freezing and thawing cycles, the samples were flushed clean, visual observations of surface conditions were recorded, the calcium chloride solution was replaced, and the test was continued. Visual ratings of the surface condition were made after 5, 10, 15, 25, and 50 cycles according to the following scale:

Table 7.2: Scaling Resistance Visual Ratings

Rating	Condition of Surface
0	no scaling
1	very light scaling (3.2-mm depth max, no coarse aggregate visible)
2	slight to moderate scaling
3	moderate scaling (some coarse aggregate visible)
4	moderate to severe scaling
5	severe scaling (coarse aggregate visible over entire surface)

Scaling resistance test results are quite subjective in nature since they are based on visual observations and the generic descriptors listed in Table 7.2. This test method is typically used in comparative analyses to determine the effects of differing materials or mix designs on PCC freezing and thawing performance. This study did not attempt to address the issues of variable materials or mix designs since these variables are less prevalent for RMP. Also, the RMP test results had to be adjusted from the standard rating scheme listed in Table 7.2 because the coarse aggregates are naturally exposed on the surface once the high water content grout cures and recedes to a certain extent between the large aggregates at the surface. With these considerations in mind, the RMP samples were tested, and the visual ratings given at the appropriate intervals were recorded as listed in Table 7.3. Figure 7.2 shows the moderately-scaled RMP samples after 50 freezing and thawing cycles.

Table 7.3: Scaling Resistance Test Results

Freezing and Thawing Cycles	Rating
5	0
10	1
15	1
25	2
50	3

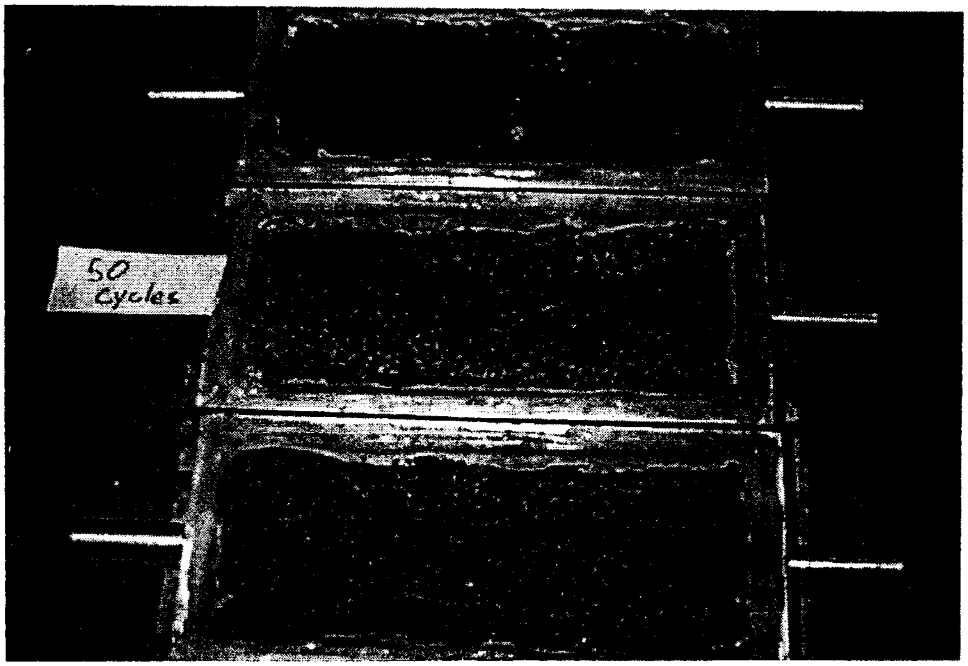

Figure 7.2: Moderate scaling of RMP samples after 50 freezing and thawing cycles

The test results indicate that the RMP may have moderate scaling under the severe freezing and thawing conditions simulated by the ASTM C672 test method. These test results can be compared to one of the largest collections of scaling resistance test data, which was reported by researchers for the Canadian Natural Resources in Ottawa, Canada (Bilodeau and Malhotra 1997). Scaling resistance tests of four different PCC mixtures containing various amounts of supplementary cementing materials were conducted and compared to the scaling resistance of standard PCC samples. Scaling resistance ratings of the experimental mixtures generally ranged from 1 to 5. The standard PCC mixtures generally rated in the 2 to 3 range, where these and all other experimental mixtures rating 3 or less were considered to have a satisfactory resistance to freezing and thawing scaling. By these standards then, RMP would appear to have a satisfactory resistance to scaling. It is important to note that RMP field experience does not indicate a tendency for freezing and thawing scaling, which may support the views of some researchers (Litvan, et al 1980) who feel that this test is too severe in comparison with actual field conditions.

Chapter 8: Traffic-Related Properties

The true value of any pavement system is ultimately measured by how well it carries traffic. Effectiveness in this regard is measured from two points of view: does the traffic adversely affect the pavement system, and does the pavement system adversely affect traffic. The first issue generally relates to load-induced damage. Based on field observations and other known engineering properties, the critical traffic load response mechanism needing to be characterized for RMP is it's fatigue characteristics. The critical issue in regards to how pavement traffic may be affected by RMP surface characteristics is skid resistance. Tests were conducted to address both the fatigue and skid resistance issues, and this part of the study is described in the following sections of this chapter.

FATIGUE CHARACTERISTICS

The fatigue characteristics of RMP were measured using standard laboratory test methods for asphalt concrete. This approach was used since RMP does have a semi-flexible, visco-elastic nature, and since this type of fatigue data is used in the layered elastic pavement design procedure selected for design analysis in this study. Fatigue test samples were produced at WES and tested at the University of California-Berkeley, where researchers and state-of-the-art equipment were leading the United States pavement industry in this type of asphalt concrete fatigue testing.

Beam samples of RMP were produced and cured using laboratory standard materials and production methods as previously described. The beam samples were originally cast in 75-mm-deep by 150-mm-wide by 380-mm-long dimensions. Each beam was removed from its mold two days after grouting and then saw-cut down the longitudinal center to produce two 75-mm by 75-mm by 380-mm beam samples. The remaining three longitudinal sides were then trimmed to produce the final 50-mm deep by 63-mm-wide by 380-mm-long dimensions required by the beam fatigue testing equipment.

The beam fatigue test method used was the same method developed for the Strategic Highway Research Program A-404 study, which is described in the literature by Monismith (Monismith 1994). The test method is generally described as a third-point, controlled-strain, beam fatigue test, with beam failure identified as the point at which the beam sample reaches a 50 percent reduction in stiffness during testing. Experience with this test method has shown that at 50 percent reduction in stiffness, micro-cracking has occurred within the material to the point where further loading will cause a rapid loss in stiffness as a result of full-depth cracking. A schematic of the flexural beam fatigue test apparatus used is shown in Figure 8.1.

Test scheduling limited the number of beam samples that could be tested, and this lead to the selection of three test temperatures and two strain levels for the fatigue test matrix. Two test temperatures were considered to be standard for this test method: 20 deg C and 30 deg C. A third test temperature at 5 deg C was selected to encompass the low pavement temperature region. Based on the RMP stiffness data produced by this study and the experience of the researchers

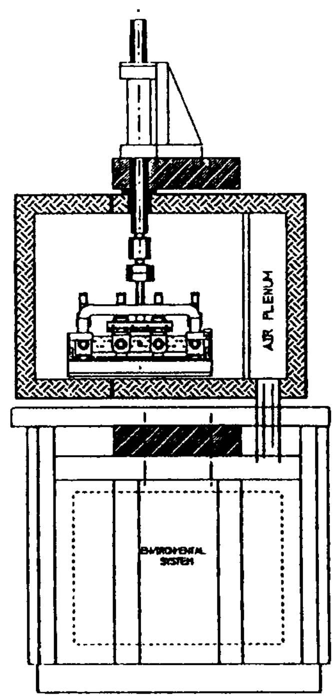

Figure 8.1: Schematic of flexural beam fatigue test apparatus (Monismith 1994)

conducting the tests, strain levels of 0.00025 and 0.00040 were selected to provide data in the 10,000 to 200,000 cycles-to-failure range.

The RMP beam fatigue test results are given in Table 8.1 and graphically displayed with regression lines in Figure 8.2. These data indicate a fatigue life versus temperature relationship similar to typical asphalt concrete (Figure 8.3). Both RMP and asphalt concrete have longer fatigue lives at higher pavement temperatures than they do at lower pavement temperatures. The fatigue test data at 5, 20, and 30 deg C were used to build a series of fatigue curves in the pavement temperature range of 0 to 40 deg C. These fatigue curves would be

Table 8.1: RMP Flexural Beam Fatigue Test Results		
Test Temperature (°C)	Average Strain (X 10^{-4})	Cycles to Failure (Nf)
5	2.50	133,890
5	2.50	145,203
5	4.00	8,518
5	4.00	4,722
20	2.50	176,111
20	2.50	149,999
20	4.00	14,382
20	4.00	13,630
40	2.50	176,615
40	2.50	149,999
40	4.00	30,768
40	4.00	23,689

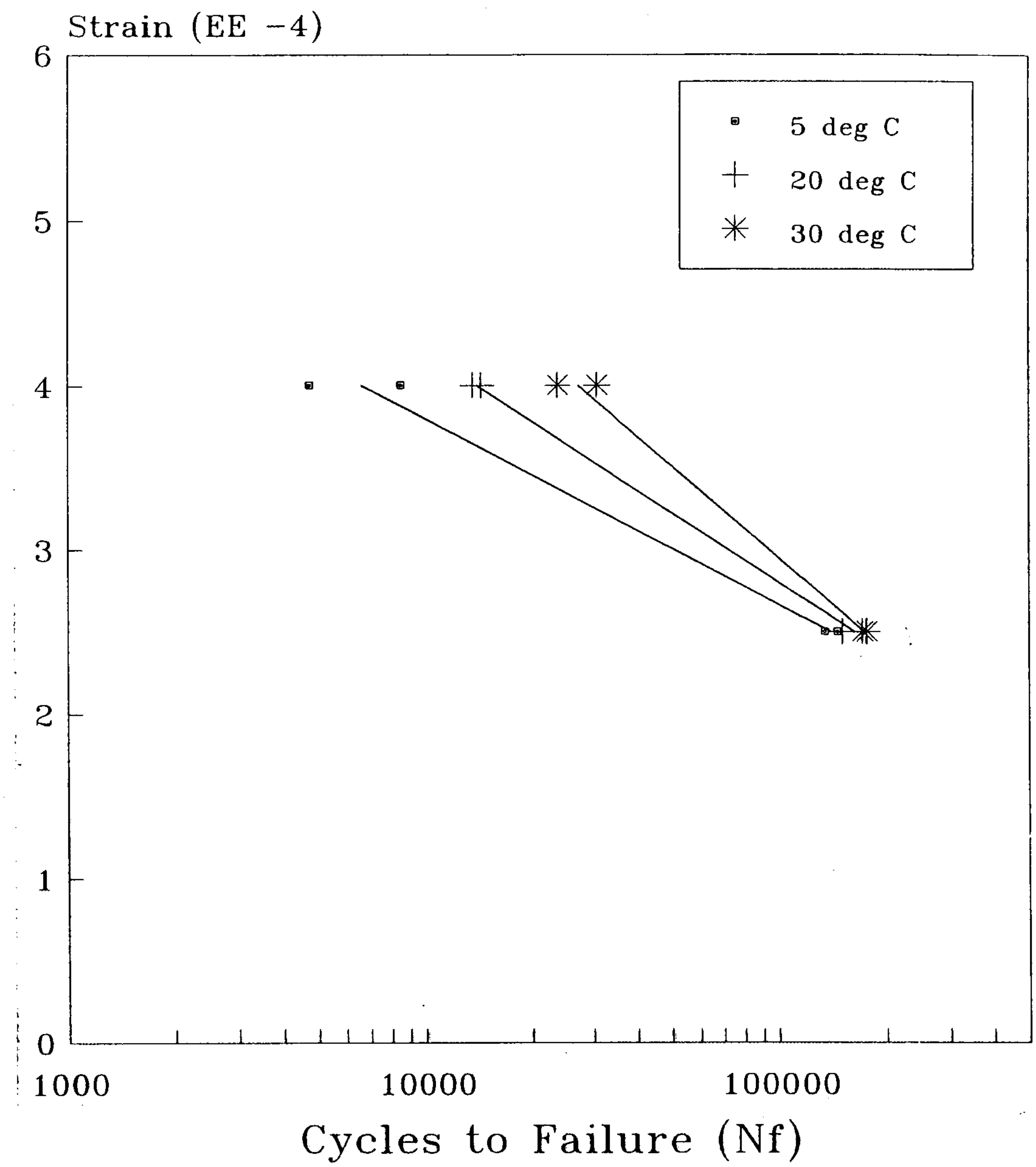

Figure 8.2: RMP fatigue curves at three test temperatures

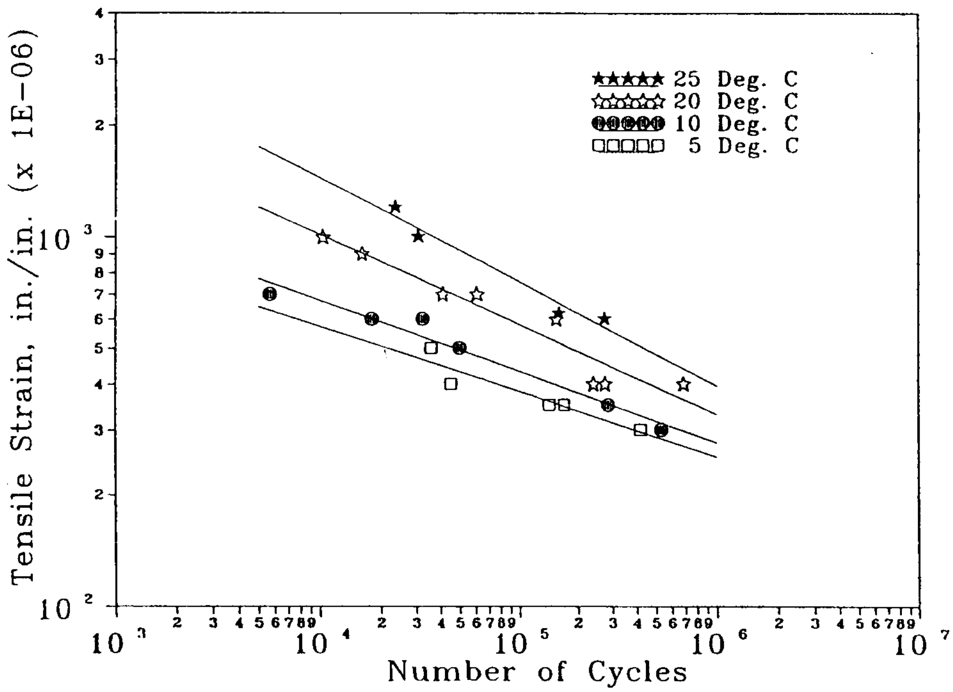

Figure 8.3: Typical fatigue versus temperature relationship for asphalt concrete (Monismith 1994)

used in RMP thickness design by allowing for an estimation of fatigue life for any calculated strain level at any given pavement temperature. The temperature-fatigue curves were produced based on the near-linear relationship between temperature and log of fatigue life at each of the two tested strain levels, as shown in Figure 8.4. The resulting fatigue curves for RMP at five pavement temperatures are shown in Figure 8.5. The application of these fatigue relationships to RMP thickness design and analysis is presented in the following chapter.

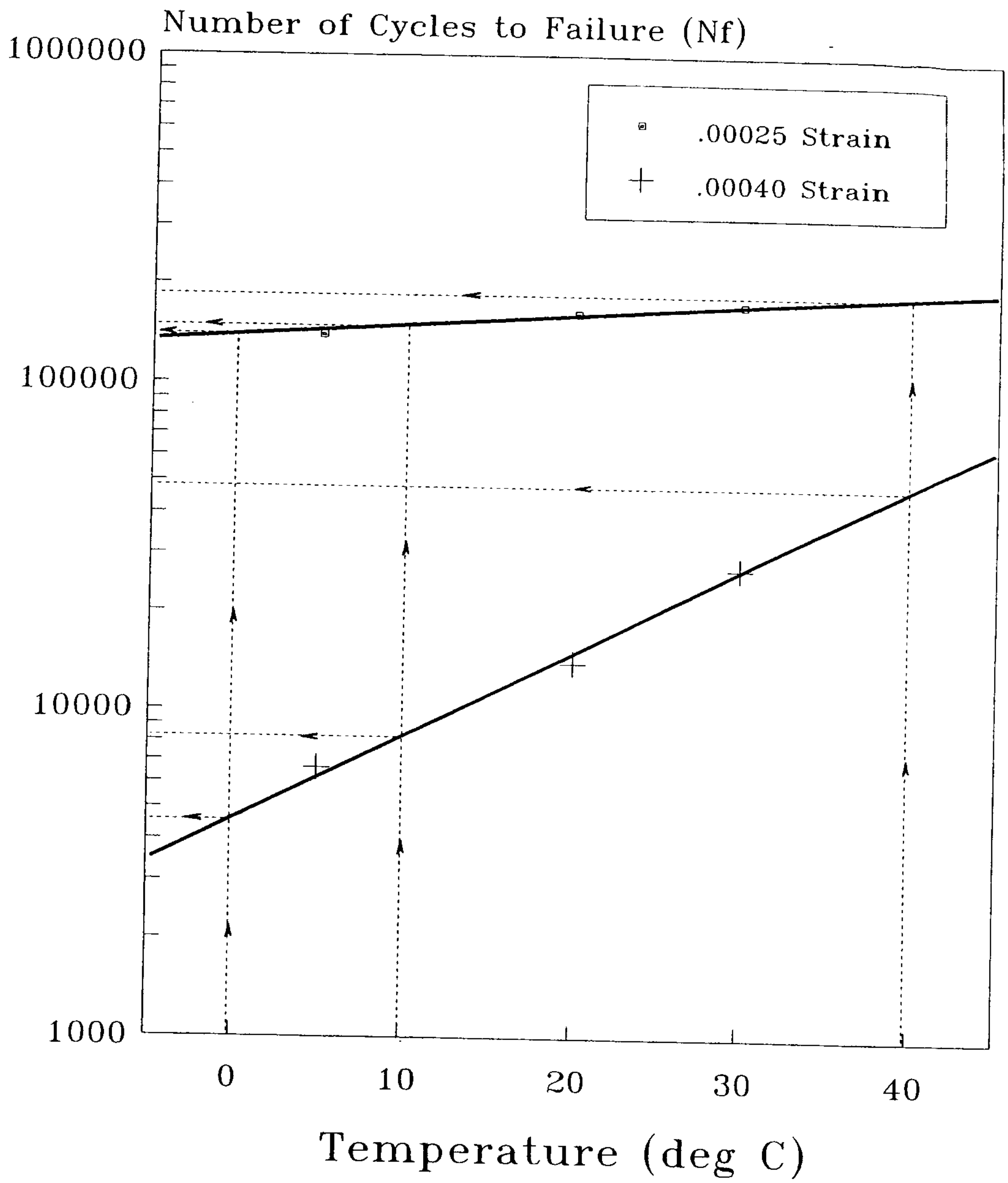

Figure 8.4: Extrapolation of fatigue data to produce additional temperature curves

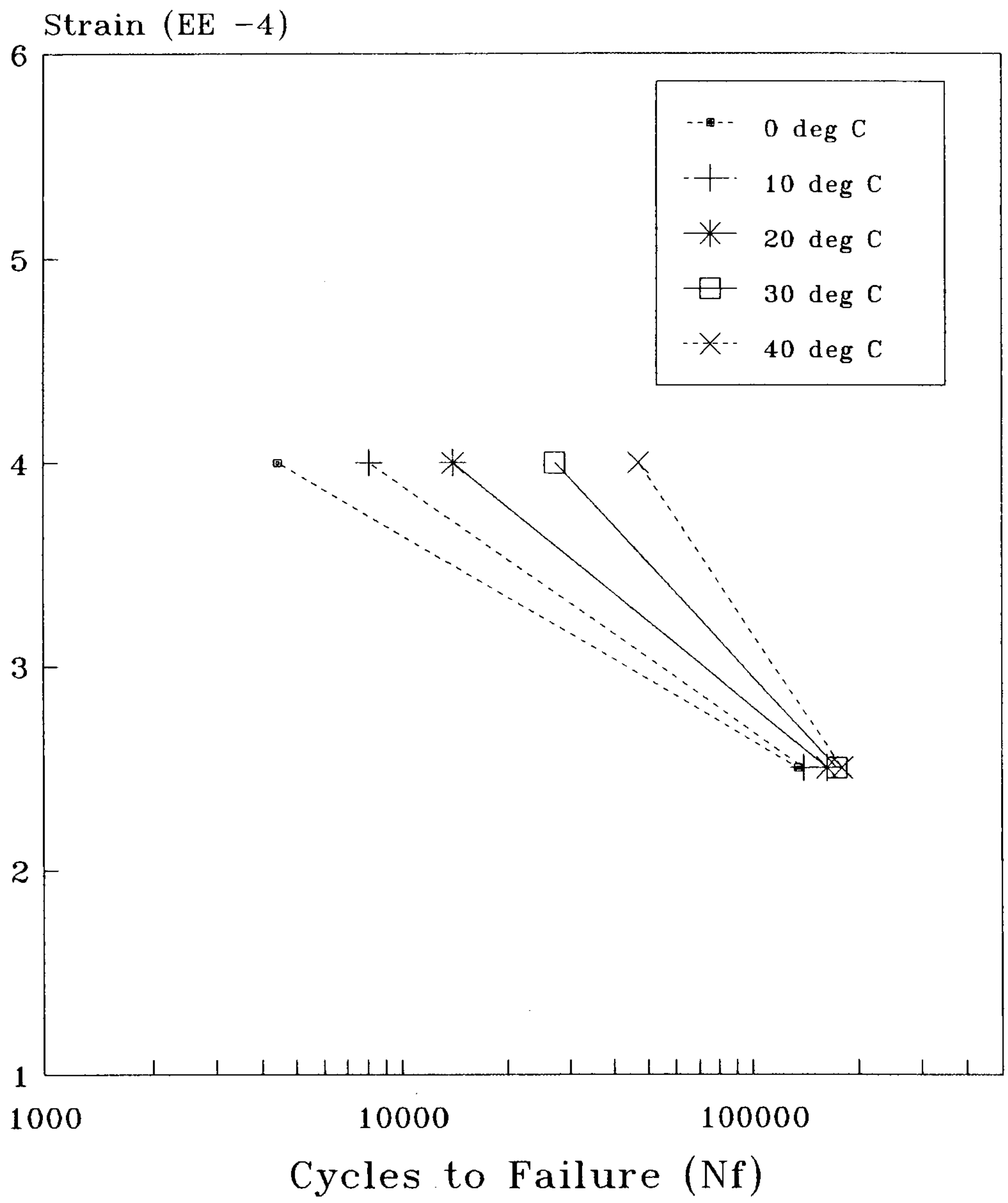

Figure 8.5: RMP fatigue curves for typical pavement temperatures

SKID RESISTANCE

From the time of the first RMP application in the United States, skid resistance has been a significant uncertainty and an important reason why many agencies have been hesitant to consider an RMP alternative. Although the surface macro-texture is relatively rough for a pavement surfacing, the surface grout appears to have a relatively smooth micro-texture. With no physical data to quantify the RMP skid resistance, applications in the United States have been limited to low-speed traffic areas.

Friction tests on RMP airfield aprons at Pope Air Force Base (AFB), North Carolina were conducted to resolve the uncertainties surrounding the issue of skid resistance. The Pope AFB site was selected for a number of reasons. When the RMP aprons were built in 1994, several other airfield aprons and taxiways were reconstructed with asphalt concrete (AC) and portland cement concrete (PCC) using the same aggregate supply as was used in the RMP. This would allow for a direct comparison between the skid resistance of RMP and that of nearby AC and PCC pavements with virtually the same age, traffic conditions, and coarse aggregates in each pavement material. Additionally, the layout of the RMP aprons at Pope AFB offered enough space to obtain some high-speed skid resistance data, whereas many other RMP sites where too small for the test vehicle to safely reach the required speeds (96 km/hr or 60 mi/hr).

The skid tests were conducted at Pope AFB from October 29 to October 31, 1996. The RMP at this site was approximately 27 months old at the time of the skid evaluation. Skid resistance was measured using a self-watering Mark V

Mu-Meter, which provides a wet surface coefficient of friction. This method of measuring pavement skid resistance is standard for the U.S. Air Force and the Federal Aviation Administration (FAA). The standard test speeds used by both of these agencies are 65 km/hr (low-speed test) and 96 km/hr (high-speed test). Numerous skid tests were conducted at both of these test speeds on airfield pavements of the same age and surfaced with RMP, AC and PCC.

The Mu-Meter used to determine pavement friction coefficient is shown in Figure 8.6. It generally consists of a small trailer with two friction-measuring wheels and a rear wheel that measures distance traveled. The Mu-Meter determines side-force friction by measuring the force created against the two friction-measuring wheels which are toed-out at a preset angle from the travel

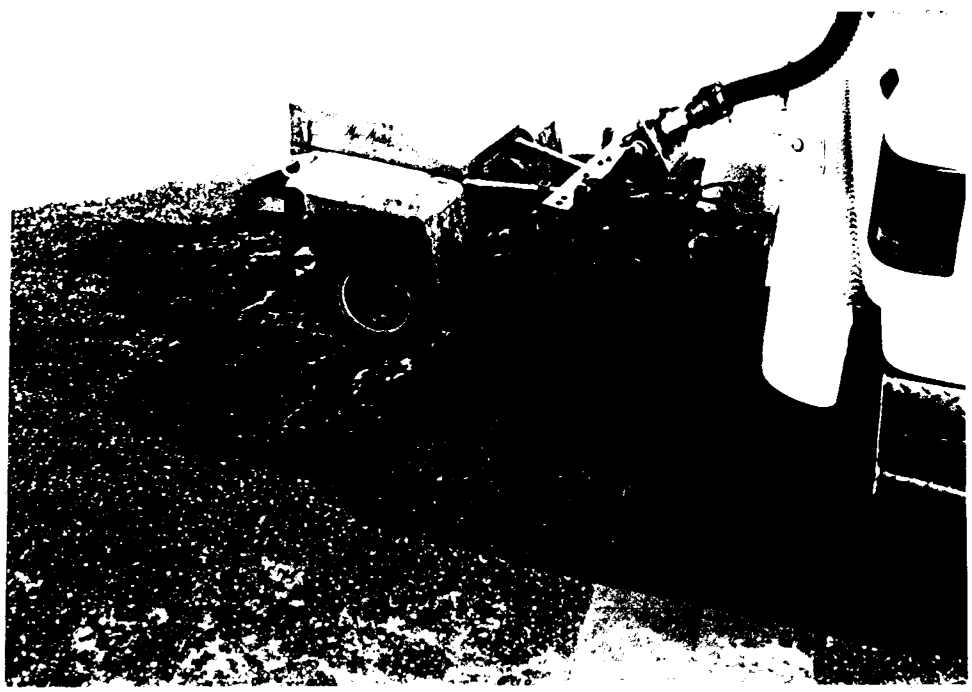

Figure 8.6: Mark V Mu-Meter used to measure RMP friction coefficient

path. This force, measured by an electronic load cell, is analyzed by a computer mounted in the cab of the tow vehicle and plotted on a strip chart showing a continuous trace of friction values for each linear foot of pavement traveled. An onboard self-watering system regulates flow from a 350-gallon tank to nozzles that distribute a 1-mm film of water beneath each wheel during the test. Friction coefficients are recorded in a range of 0.00 to 1.00, with higher values representing greater skid resistance. Figure 8.7 depicts a 96 km/hr friction test on an RMP apron at Pope AFB.

The skid evaluation test results are given in Table 8.2 along with the in-service performance standards specified by the FAA (Federal Aviation Administration 1991) and adopted by the U.S. Air Force. In general, these skid

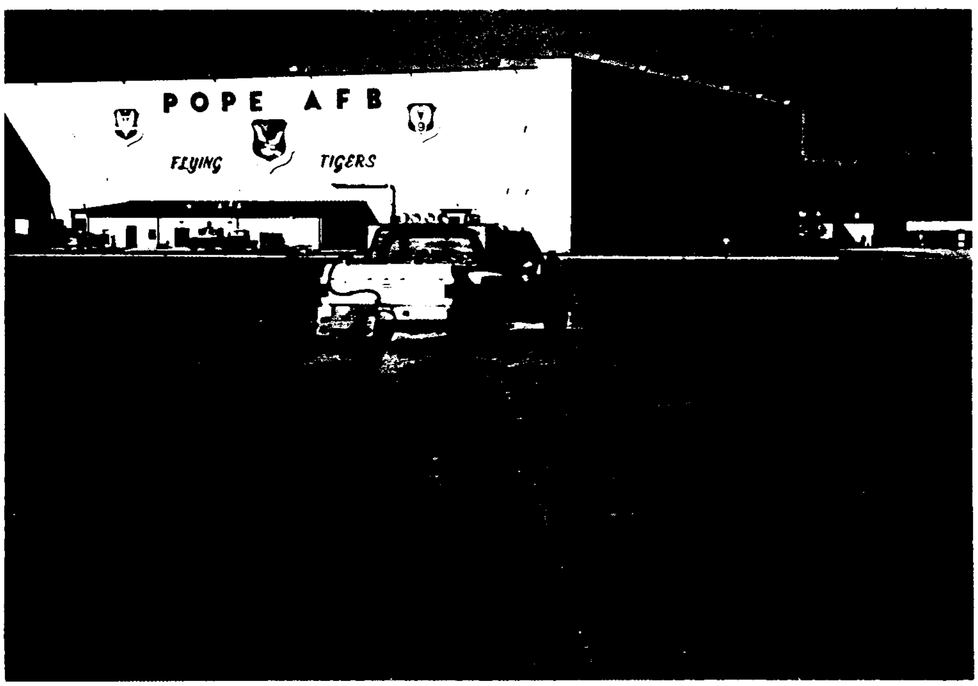

Figure 8.7: High-speed friction test being performed on RMP at Pope AFB

Table 8.2: Comparative Skid Test Results from Pope AFB					
		Low-Speed Test		High-Speed Test	
Pavement Surfacing	Location	Friction Coefficient	Skid Potential *	Friction Coefficient	Skid Potential*
RMP	SB Apron	0.58	Low	0.53	Low
RMP	O Apron	0.57	Low	0.43	Moderate
RMP	H6 Apron	0.58	Low	No Test	----
PCC	Taxiway A	0.48	Moderate	0.32	Moderate
PCC	Taxiway HH	0.62	Low	No Test	----
AC	Taxiway G	0.60	Low	No Test	----
AC	Taxiway GK	0.60	Low	No Test	----
* U.S. Air Force Standards					

test results indicate that the RMP at Pope AFB has a relatively low skid potential, with both low-speed and high-speed friction coefficients that are quite acceptable by current U.S. Air Force and FAA standards. The RMP friction coefficients were considerably higher than the friction coefficient of one PCC pavement and marginally lower than another PCC pavement when tested at 65 km/hr. At the same speed, the RMP had about the same coefficient as both AC taxiways. At the higher test speed of 96 km/hr, the RMP had friction coefficients well above that of the PCC taxiway tested. Unfortunately, no high-speed tests of the AC-surfaced taxiways could be obtained because of the lack of surface area needed to gain speed. It is clearly apparent, however, that RMP compares quite favorably with both AC and PCC pavements at this site in terms of skid resistance. There was nothing unusual about the surface condition of any of the pavements tested at this site to prevent a valid comparative analysis between the three types of pavement surfacings.

Visible evidence from older RMP sites indicates that the surface grout slowly wears off of the surface of the coarse aggregates for a number of years after placement. This gradual wearing away of surface grout had just begun at Pope AFB. As more of the surface grout is removed by traffic, it is expected that skid resistance will gradually improve until leveling off once the majority of the surface grout is gone. The improved skid resistance of aged RMP should remain fairly consistent, given a durable aggregate that is not susceptible to polishing under traffic. This reasoning implies that the good skid resistance measured for the RMP at Pope AFB should improve at least slightly with time.

Chapter 9: Linear Elastic Layer Modeling

The current design approach used for RMP in the United States and in other countries can be described as purely empirical in nature. Simply stated, an RMP design allows for any traditional flexible pavement design approach a given agency may use for the given conditions and materials, and then the top 50-mm of asphalt concrete surfacing is replaced with 50-mm of RMP. Although this design approach is simple and appears to provide adequate thickness for acceptable early performance, there is a real need for a more sound, mechanistic design approach that will help to prevent dangerous "under-designs" as well as costly "over-designs." RMP appears to be well-suited for a wide range of traffic and climatic conditions, making a mechanistic approach even more important.

Physical evidence from early RMP field performance coupled with the engineering properties determined by this and other studies indicate that load-induced cracking is the critical failure mode. These load-induced cracks can logically result from three scenarios: excessive vertical strains at the top of the pavement subgrade, resulting in damage from multi-layer pavement deflections; excessive horizontal strains at the bottom of the asphalt concrete, which would induce cracks that would translate upwards into the RMP layer; and excessive horizontal strains at the bottom of the RMP surfacing, which would result in progressive cracks in the RMP. The most appropriate method for modeling these strains under various loads, environmental conditions, layer thicknesses, and material properties is the linear elastic layer method for flexible pavement design.

This is the design method of choice for the agencies likely to build the majority of the near-future RMP projects, making the linear elastic layer method the most practical choice as well.

CORPS OF ENGINEERS LAYERED ELASTIC DESIGN METHOD

The Corps of Engineers Layered Elastic Design (COE-LED) Method allows for the structural design and analysis of flexible pavement systems by characterizing each pavement layer in terms of its stiffness, applying vertical loads representative of the projected traffic, and then predicting stresses and strains at critical locations within the pavement structure. Limiting the maximum vertical compressive strain at the top of the subgrade layer is used to prevent surface damage from excessive deformation in the subgrade layer. Limiting the horizontal tensile strain at the bottom of the asphalt concrete surfacing is used to prevent load-induced surface cracking. Use of a cumulative linear damage concept permits the rational handling of variations in the asphalt concrete properties and subgrade strength caused by cyclic climatic conditions. The limiting strain criteria for both the subgrade and asphalt concrete were developed through extensive laboratory and test section evaluations made by the U.S. Army Engineer WES (Chou, 1977). The strains relating to the pavement's response to imposed traffic loads are computed by the use of Burmister's solution for multilayered elastic continua. There are many details describing the Corps of Engineers approach to designing flexible pavements by the elastic layered method that are not described here, but are found in the appropriate military technical manual (Departments of the Army and the Air Force 1989). A simplified flow chart of the COE-LED method is shown in Figure 9.1.

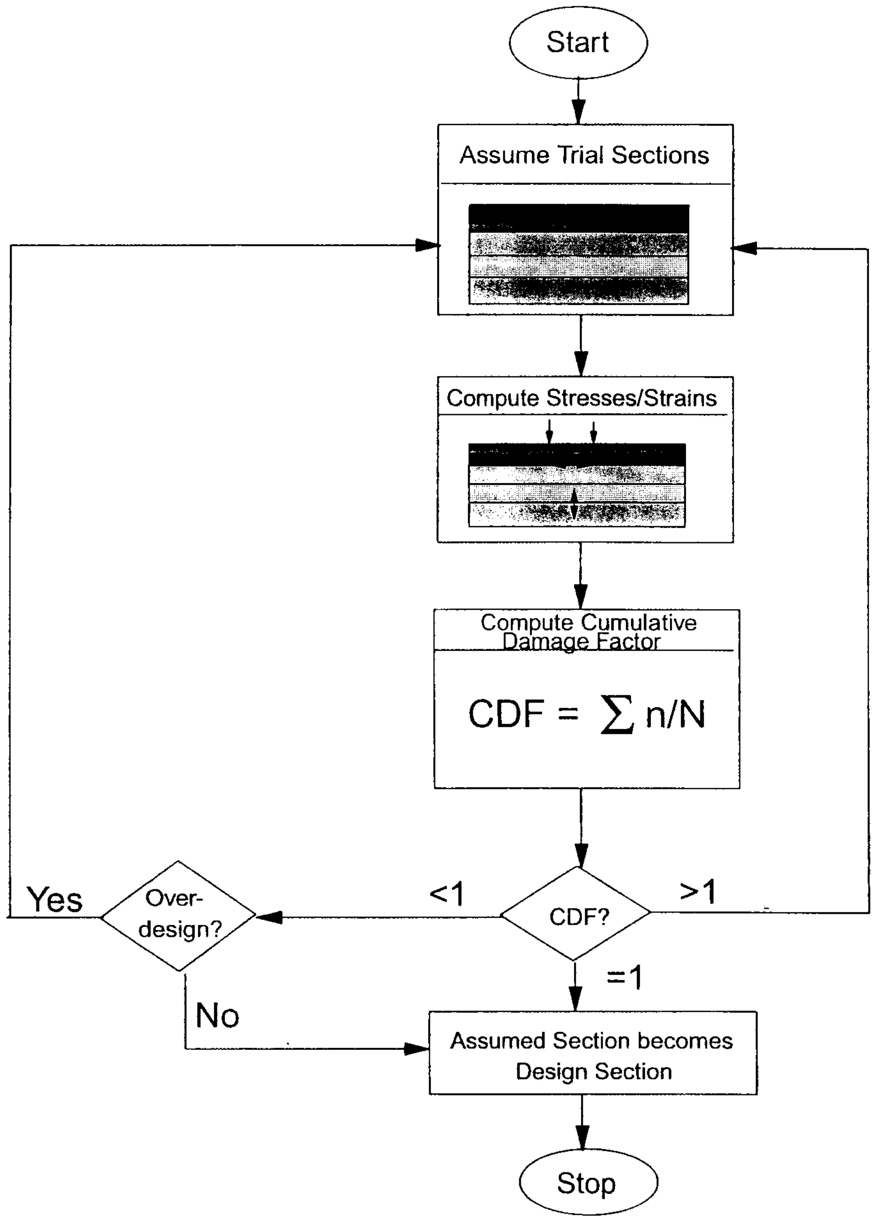

Figure 9.1: Flow chart of COE-LED method for flexible pavements

Computer codes and supporting databases have been written by researchers at WES in recent years to simplify and speed up the design input and computational requirements of a layered elastic flexible pavement design. These computer programs are used throughout the federal government and by many other agencies world-wide, as they are readily available to the general public. When using these programs, the following assumptions are made:

1. The pavement is a multilayered structure, and each layer is represented by a modulus of elasticity and Poisson's ratio;
2. The interface between layers is continuous; i.e., the friction resistance between layers is greater than the developed shear force;
3. The bottom layer is of infinite thickness;
4. All loads are static, circular, and uniform over the tire contact area.

A Corps of Engineers layered elastic pavement analysis computer program known as WESPAVE was used to determine the suitability of the Corps of Engineers layered elastic strain criteria and pavement response models for an RMP design. The WESPAVE program is typically used for airfield flexible pavement design and analysis. The program requires input data on the pavement structure (individual layer thickness, modulus, and Poisson's ratio) and on the projected traffic (aircraft type, load, and number of design coverages). Gear configurations, tire contact areas, tire pressures, and other aircraft-specific data are built into the computer's traffic database for ease of use. The program outputs include the maximum strains at the critical locations within the pavement structure and the resulting projections of allowable numbers of aircraft coverages based on either subgrade or asphalt concrete strain criteria.

RMP LAYERED ELASTIC MATERIAL PROPERTIES

There are two RMP material properties required for a layered elastic design analysis: stiffness and fatigue. A stiffness versus temperature curve would allow the RMP designer to input seasonal effects. These data were produced by the resilient modulus tests described earlier in this report. Since there was some data scatter at each temperature, there is some question about where the stiffness design curve should be. The data presented previously in Figure 6.3 indicated a much smaller data scatter for RMP stiffness when compared to typical asphalt concrete stiffness values. This relatively small range makes the idea of using a single stiffness versus temperature curve more feasible. Also, different combinations of traffic load, layer thicknesses, and underlying support can make a higher RMP stiffness value slightly over-conservative in one case and slightly under-conservative in another. All of this evidence supports the feasibility of using a single RMP stiffness versus temperature curve produced from the statistical mean values of the laboratory and field data previously reported in Table 6.1. This design input curve is given in Figure 9.2. As previously discussed in Chapter 6, a Poisson's ratio value of 0.27 is appropriate for RMP design at virtually any pavement temperature.

The flexural beam fatigue data presented and discussed in Chapter 8 of this report provides the basis for determining the suitability of a particular RMP

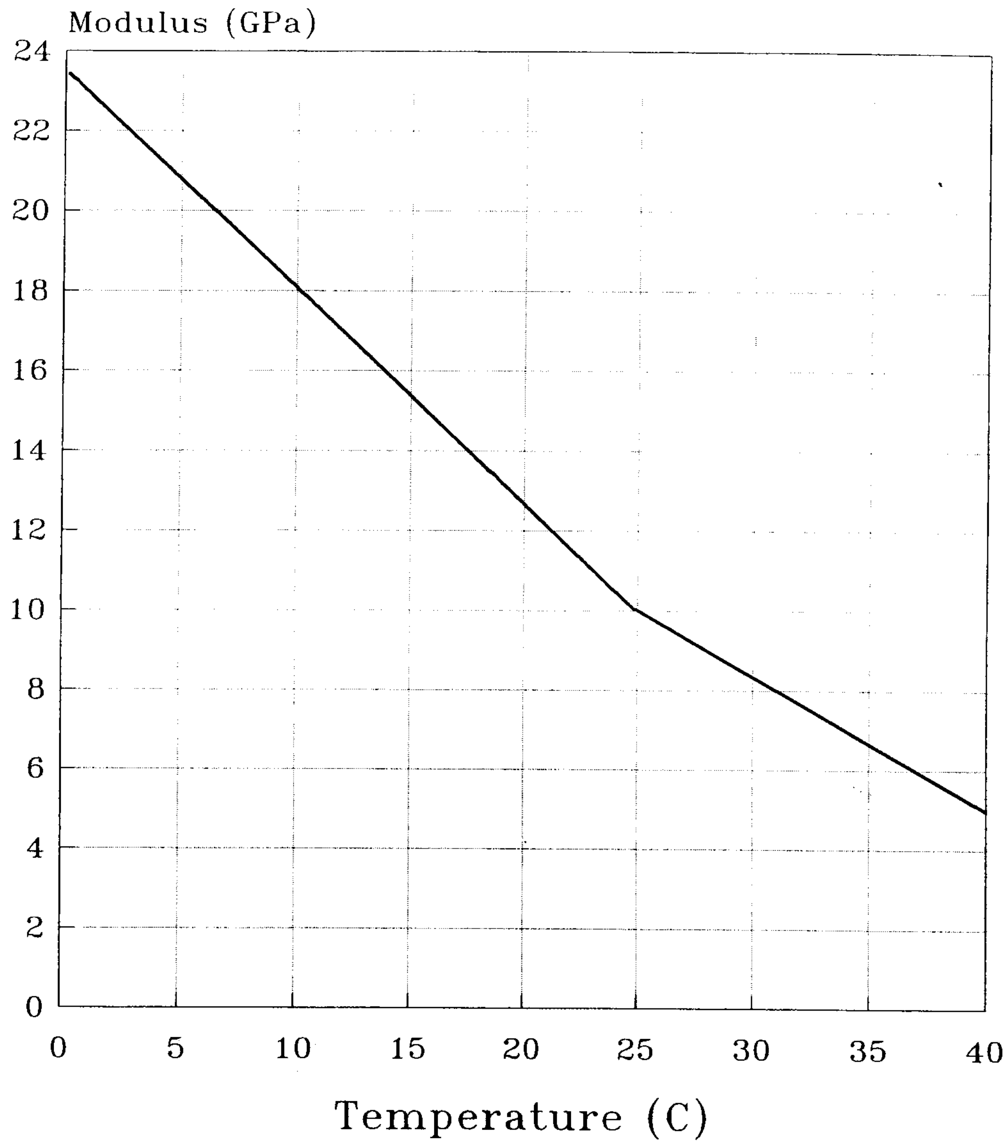

Figure 9.2: RMP resilient modulus versus temperature design curve

design section. It is important to note that experience with similar fatigue data for asphalt concrete has indicated that laboratory fatigue tests tend to underestimate fatigue life in the field. Pavement researchers have attempted to account for this discrepancy by applying shift factors, which have varied from slightly more than 1 to more than 400 (Matthews, et al 1993). Two of the more prominent reasons for the conservatism of laboratory fatigue data are accounted for in the Corps of Engineers Layered Elastic Design Method: temperature variability and traffic wander. Only more field experience and time will help to resolve the validity of the RMP fatigue relationships, but until such future validations can be made, the unresolved conservatism assumed by the laboratory fatigue data is an acceptable flaw for a first-generation design method.

The fatigue curves for various pavement temperatures presented previously in Figure 8.5 were used to generate the RMP design fatigue curves shown in Figure 9.3. Extension of the fatigue regression lines into the larger strain range is a common technique used to minimize the required laboratory testing time (Tayebali, et al 1996). The designer should realize, however, that the reliability of the fatigue data within the strain range tested is assumed to be higher than that in the extrapolated high-strain range. Since virtually all of the fatigue curves are in the 10^{-4} strain level for the practical 100 to 500,000 load cycle range, it can be said that strains at or above the 10^{-3} level are likely to cause very quick failures and strains at or below the 10^{-5} level are negligible in terms of fatigue damage.

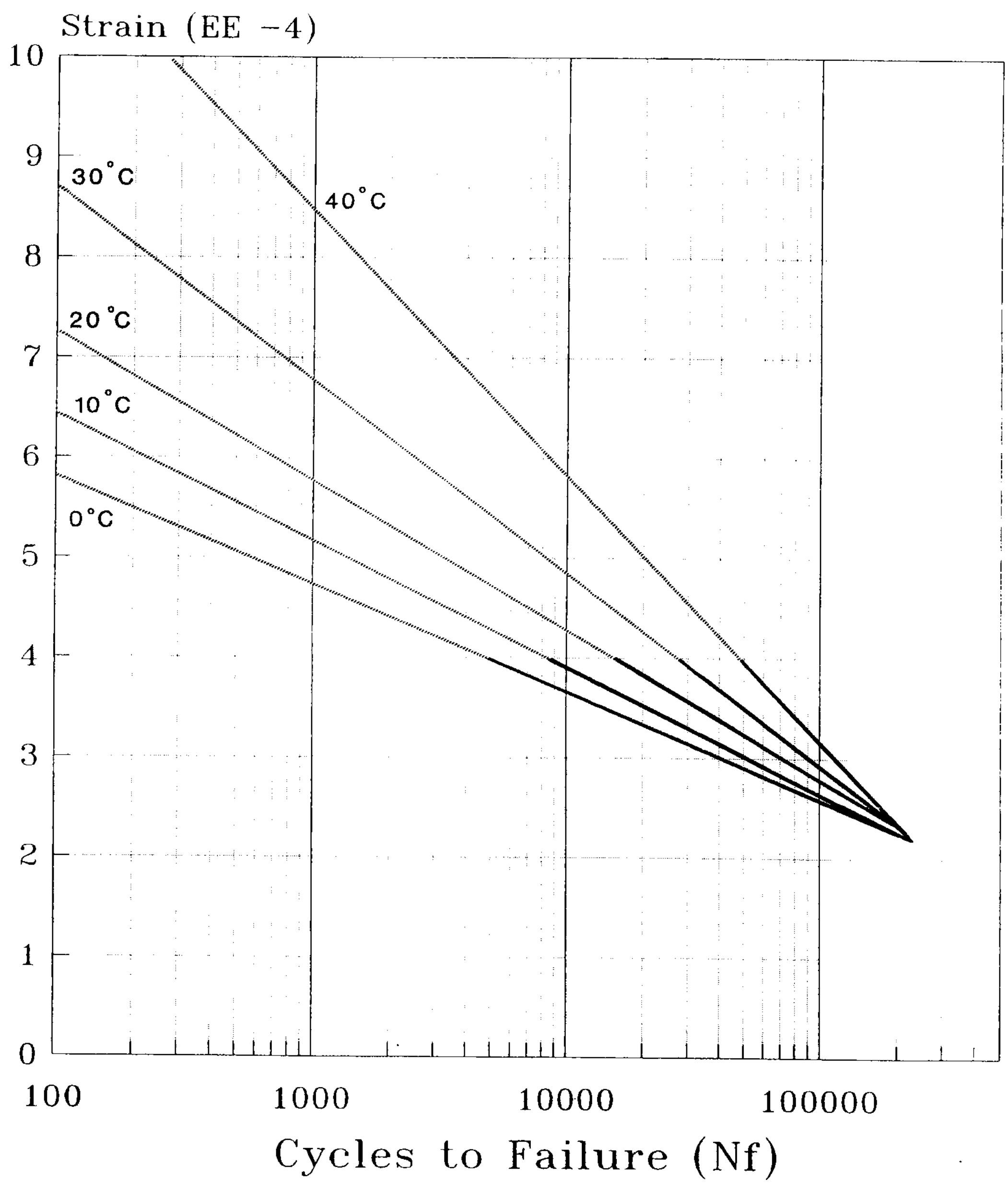

Figure 9.3: RMP fatigue design curves at various pavement temperatures

RMP STRUCTURAL DESIGN EXAMPLE

A hypothetical RMP airfield apron design example is presented here to demonstrate the feasibility of using the Corps of Engineers Layered Elastic Design Method for RMP structural design. For comparison, an optimum asphalt concrete design and various RMP designs were made. Non-SI (English) units are used with the data for this example since the current WESPAVE computer program is designed for these units.

The airfield site is assumed to be in Shreveport, Louisiana where an Army Class III airfield apron is to be designed for 200,000 passes of a C-130 aircraft with a design load of 155,000 lb. The modulus values for the subgrade, subbase, and base materials are assumed to be 10,000 psi, 25,000 psi, and 50,000 psi, respectfully. Subgrade CBR is assumed to be 6. The asphalt concrete (AC) to be used at this site was tested and has a modulus versus temperature relationship as shown in Figure 9.4. Standard Poisson's ratios for the AC, granular base, subbase, and cohesive subgrade are 0.35, 0.30, 0.30, and 0.40, respectfully. AC materials are assumed to cost more than base materials, which are in turn assumed to cost more than subbase materials. From the climatic data of this site, the design pavement temperatures are obtained and the design AC modulus values are determined as shown in Table 9.1. To reduce the number of computations, the 12 month groups are reduced to four seasonal groups as shown in Table 9.2.

By using the appropriate aircraft design curve found in the U.S. Army Technical Manual TM 5-825-2 (Departments of the Army and Air Force 1989), the total thickness of pavement required for the design aircraft and the 6 CBR

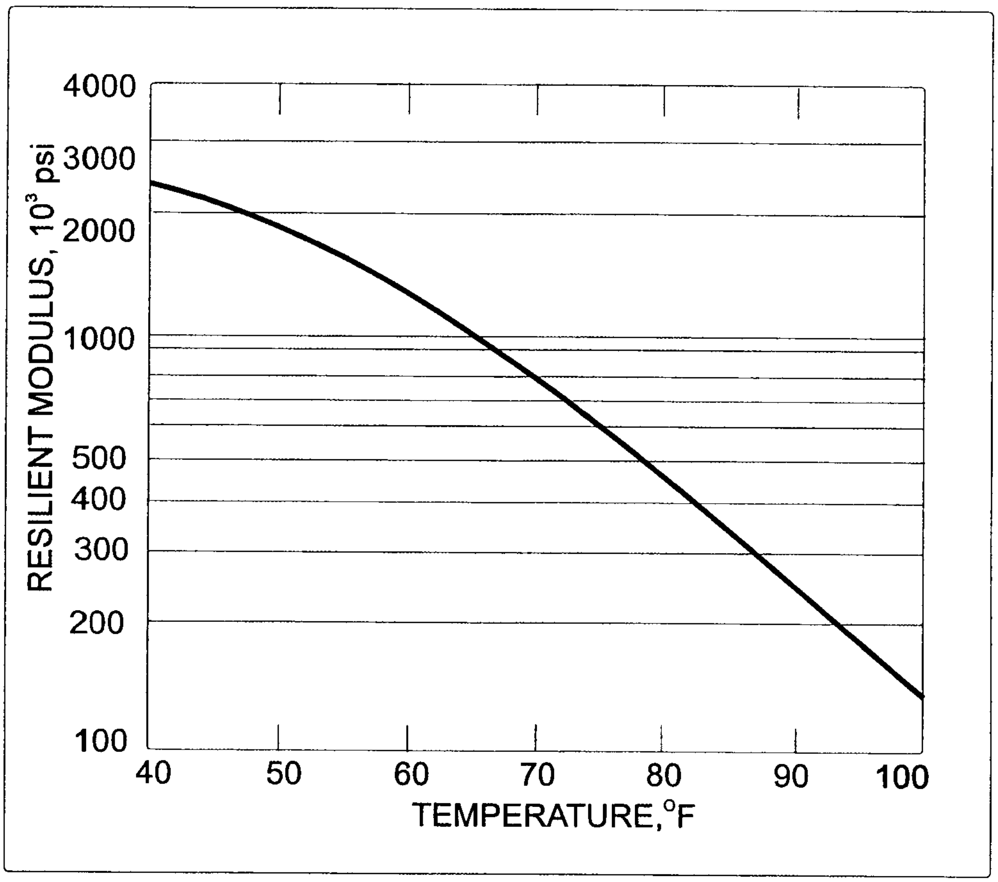

Figure 9.4: Temperature-modulus relationship for design example AC

Table 9.1: Monthly Design Pavement Temperatures and AC Moduli

Month	Pavement Design Temperature (°F)	Resilient Modulus (10^3 psi)
Jan	56	1500
Feb	60	1270
Mar	67	920
Apr	76	570
May	84	360
Jun	92	220
Jul	95	180
Aug	95	180
Sep	89	260
Oct	77	540
Nov	65	1000
Dec	57	1400

Table 9.2: Grouping Traffic into Seasonal Traffic Groups					
		Resilient Modulus (10^3 psi)			
Group	Month	Monthly Value	Group Average	Percent of Total Traffic	Group Required Passes (N_{reqd})
	Jan	1500			
1	Dec	1400	1390	25.0	50,000
	Feb	1270			
2	Nov	1000			
	Mar	920	960	16.7	33,400
	Apr	570			
3	Oct	540	490	25.0	50,000
	May	360			
	Sep	260			
4	Jun	220			
	Jul	180	210	33.3	66,600
	Aug	180			

subgrade is estimated to be approximately 28 in. Since U.S. Army standards require a minimum AC thickness of 5 in. and a minimum base course thickness of 6 in. for an airfield apron, the initial design section is as follows: 5 in. of AC; 6 in. of base; 17 in. of subbase. This would likely represent the most economical design section and if any added strength would be required, then replacement of subbase material with base material would be the first logical choice.

The WESPAVE computer program was used to determine the optimum AC design section for this design example. Optimum design was selected for this example when the most economical structural profile (minimum allowable AC and base course thicknesses) provided enough allowable passes (N_{allow}) to just exceed the required number of passes (N_{reqd}) in each of the four climatic seasons. Allowable passes were computed by the program for each season's profile based on limiting strain criteria for the bottom of the AC section and the top of the subgrade. Examples of the WESPAVE input windows for this design example are shown in Figures 9.5 and 9.6. The output file for the optimum AC design section is shown in Figure 9.7.

The optimum AC design for this design example is summarized in Table 9.3. The limiting strains for this design turn out to be for the AC during the coldest season and for the subgrade during the warmest season. These limiting strains are typical for these types of seasons.

The first application of RMP data to this design example is to check the suitability of the current design approach. This is done by simply replacing the top 2 in. of AC surfacing with 2 in. of RMP. In addition to the AC and subgrade strain evaluations, the strain at the bottom of the RMP layer is used with the appropriate fatigue curve from Figure 9.3 to determine the allowable number of aircraft passes based on the RMP fatigue criteria. The average pavement temperature for the given season is used to interpolate between the RMP fatigue curves. For this example, the RMP design section also represents a typical "inlay" design, where 2 in. of AC would be removed from an existing flexible pavement and replaced with 2 in. of RMP.

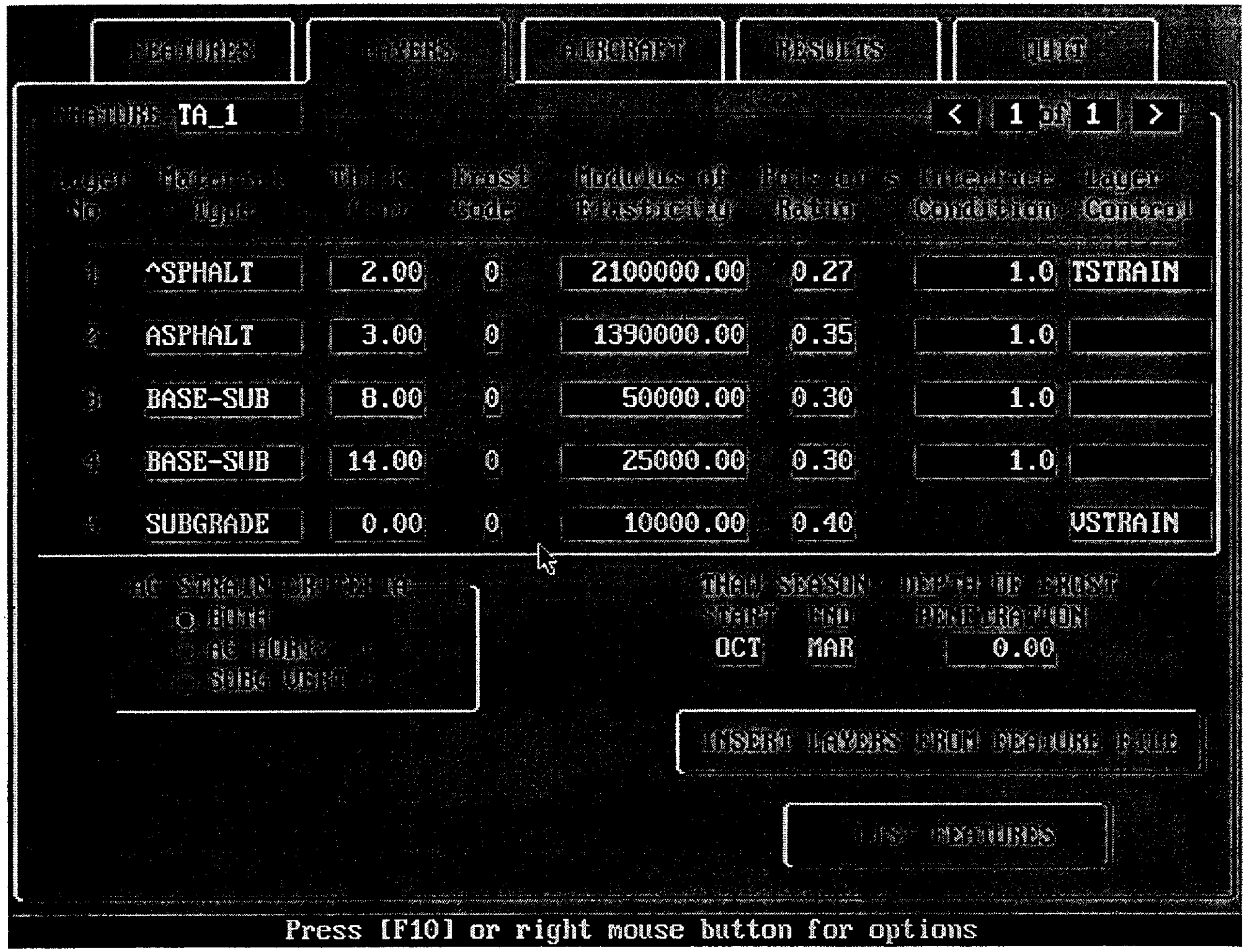

Figure 9.5: Example of WESPAVE pavement layer input window

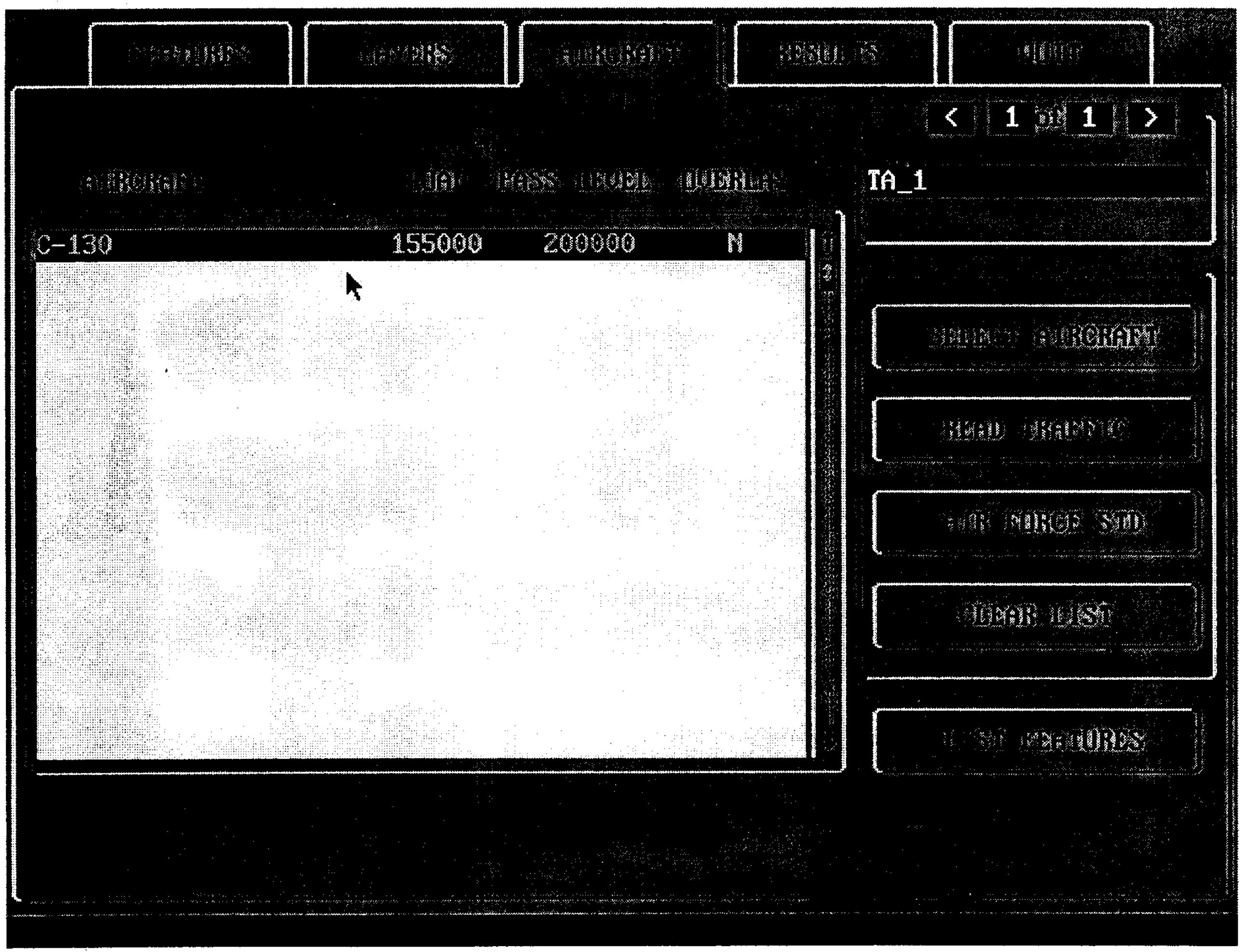

Figure 9.6: Example of WESPAVE traffic input window

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******************************************************************************
********************************"WESPAVE"*******************************
*********************EXECUTED:  8-28-1997 @ 14:13*********************

PROBLEM:   1 TITLE: TA_1

LAY                                 FROST     MODULUS, PSI        THICK.   POIS  SLIP
NO.     MATERIAL TYPE                CODE     NDT       FROST       IN.    RAT.  VALUE
*** ********************** ***** ********* ********* ******** **** *****
 1             AC                    0     1390000.  1390000.       5.00  .35     1.
 2       BASE OR SUBBASE             0       50000.    50000.       8.00  .30     1.
 3       BASE OR SUBBASE             0       25000.    25000.      14.00  .30     1.
 4          SUBGRADE                 0       10000.    10000.      10.00  .40     1.
 5          SUBGRADE                 0       10000.    10000. SEMI-INF  .40

                         **************    PCC    **************
      TRAF COND  SUBG           DES   FLX   JT                       AC   FROST
  E   AREA FACT k/CBR  EFF k SCI   STR   LRF   CB    CR    F     CRIT  PEN
----- ---- ---- ------ ----- ---- ----- ---- ---- ---- ---- ---- ------
NDT    A   1.00    6.7                                              AC     .00

                       PAVEMENT EVALUATION SUMMARY
                                                           **REQD OVERLAY **
                  PCN - NDT   PCN - THAW ALLOWABLE                       PCC
 DESIGN TRAFFIC  JAN - DEC   XXX - XXX   PASSES          AC   PCC NO BOND
-------------- ----------- ----------- ---------       ---- ---- -------
TYPE:      C-130  25/F/C/W/T              94180.  NDT
LOAD: 155.0 ACN  30/F/C/W/T
PASSES:  200000 AGL: 133.3


******************************************************************************
******************************************************************************
******* WESPAVE SUMMARY COMPLETE ******


                        BLIST FOR PROBLEM NUMBER   1
******************************************************************************
AIRCRAFT:C-130       LOAD:    155000. PASSES:     200000 MODULI:NDT

EVAL
POS      X        Y      EPS,VERT   DEPTH  EPS,HORZ    DEPTH SIG,HORZ  DEPTH
---- ------ ------ ---------- ------ ---------- ------ -------- ------
  1     .00     .00                         .2166E-03   5.00
  2   30.00     .00                         .7532E-04   5.00

LAY 1  ALLOWABLE  ALLOWABLE   ALLOWABLE  MINIMUM ALLOWABLE ALLOWABLE
THICK  AC STRAIN SUBG STRAIN PCC STRESS  RATIO   PASSES       LOAD
------ --------- ----------- ---------- ------- --------- ---------
  5.00  .186E-03                                 .86     94180.   133327.
```

Figure 9.7: WESPAVE output file for optimum AC design

Table 9.3: Summary of Optimum AC Design					
Pavement Layer	Thickness (in.)	Seasonal Modulus Values (10^3 psi)			
		Group 1	Group 2	Group 3	Group 4
AC	5	1390	960	490	210
Base	8	50	50	50	50
Subbase	14	25	25	25	25
Subgrade	----	10	10	10	10
N_{reqd}		50,000	33,400	50,000	66,000
AC Strain		2.17×10^{-4}	2.43×10^{-4}	2.73×10^{-4}	2.58×10^{-4}
AC N_{allow}*		94,180	141,967	474,648	4,000,000+
Subgrade Strain		7.66×10^{-4}	8.19×10^{-4}	9.04×10^{-4}	9.97×10^{-4}
Subgrade N_{allow}		2,069,682	980,684	320,531	104,774
* Note: Maximum computed N_{allow} is 4,000,000.					

The seasonal input and output data for the RMP inlay design are summarized in Table 9.4. The results of this particular design analysis suggest that the current RMP design approach provides a pavement structure that is more than adequate to carry the prescribed traffic throughout the design life. The significant elements of this particular design analysis include the following points:

1. The current RMP design approach provides a marginal amount of additional conservatism to the existing flexible pavement design procedure.
2. The structural capacity of the RMP layer significantly improves the potential fatigue life of the pavement subgrade during all climatic seasons.

3. The bond between the RMP and AC layers affects the strains in the AC layer most significantly during seasons with warmer temperatures, when the difference between their respective stiffnesses are greatest. The critical season for the AC strain remains during the cold months, however, when the stiffer composite pavement surfacing can withstand fewer strain repetitions before failing.
4. The typical airfield design which is suitable for full-term fatigue life when surfaced with AC provides more than enough structural support to prevent premature fatigue cracking in the RMP surfacing.

Table 9.4: Summary of RMP Inlay Design

Pavement Layer	Thickness (in.)	Seasonal Modulus Values (10^3 psi) Group 1	Group 2	Group 3	Group 4
RMP	2	2100	1775	1450	980
AC	3	1390	960	490	210
Base	8	50	50	50	50
Subbase	14	25	25	25	25
Subgrade	----	10	10	10	10
N_{reqd}		50,000	33,400	50,000	66,000
RMP Strain*		1.71×10^{-6}	-1.85×10^{-5}	3.78×10^{-6}	2.51×10^{-5}
RMP N_{allow}		Infinite	Infinite	Infinite	Infinite
AC Strain		2.18×10^{-4}	2.50×10^{-4}	3.07×10^{-4}	3.46×10^{-4}
AC N_{allow}		91,080	123,147	264,498	1,394,513
Subgrade Strain		7.30×10^{-4}	7.66×10^{-4}	8.18×10^{-4}	8.86×10^{-4}
Subgrade N_{allow}		3,595,357	2,080,231	984,048	399,561

* Note: Negative strain values indicate compression.

The final phase of this design example involves an attempt to reduce the pavement thickness in order to obtain an optimal (least costly) RMP design. This design would represent a newly-constructed RMP pavement where no pavement system previously existed, or possibly a full-depth redesign using RMP. Through the trial-and-error process of adjusting base and subbase thicknesses, an optimum RMP design was developed and is summarized in Table 9.5.

Table 9.5: Summary of Optimum Full-Depth RMP Design

Pavement Layer	Thickness (in.)	Seasonal Modulus Values (10^3 psi)			
		Group 1	Group 2	Group 3	Group 4
RMP	2	2100	1775	1450	980
AC	3	1390	960	490	210
Base	6	50	50	50	50
Subbase	14	25	25	25	25
Subgrade	----	10	10	10	10
N_{reqd}		50,000	33,400	50,000	66,000
RMP Strain*		2.67×10^{-6}	-1.85×10^{-5}	5.23×10^{-6}	2.85×10^{-5}
RMP N_{allow}		Infinite	Infinite	Infinite	Infinite
AC Strain		2.30×10^{-4}	2.67×10^{-4}	3.29×10^{-4}	3.78×10^{-4}
AC N_{allow}		69,570	88,598	187,250	896,237
Subgrade Strain		7.96×10^{-4}	8.41×10^{-4}	9.07×10^{-4}	9.93×10^{-4}
Subgrade N_{allow}		1,348,434	723,567	305,159	109,616

* Note: Negative strain values indicate compression.

The optimum full-depth RMP design allowed for a 2-in. reduction in base course thickness when compared to the RMP inlay design. Over a large pavement area, this could amount to a substantial cost savings in construction materials. The critical design point for this RMP structural design is again tensile strain at the bottom of the AC layer during the colder season.

A smaller fighter-class aircraft (F-16) was substituted into the design scenario presented here to determine if the general trends between AC and RMP designs would hold for a completely different type of traffic load. Fighter aircraft tend to affect surface layers more with their single-wheel landing gears and higher tire pressures. In general, the same trends established by the C-130 design example held true for the F-16 aircraft design. The cross-sections of the traditional AC, RMP inlay, and RMP optimum designs for the F-16 aircraft are summarized in Figure 9.8.

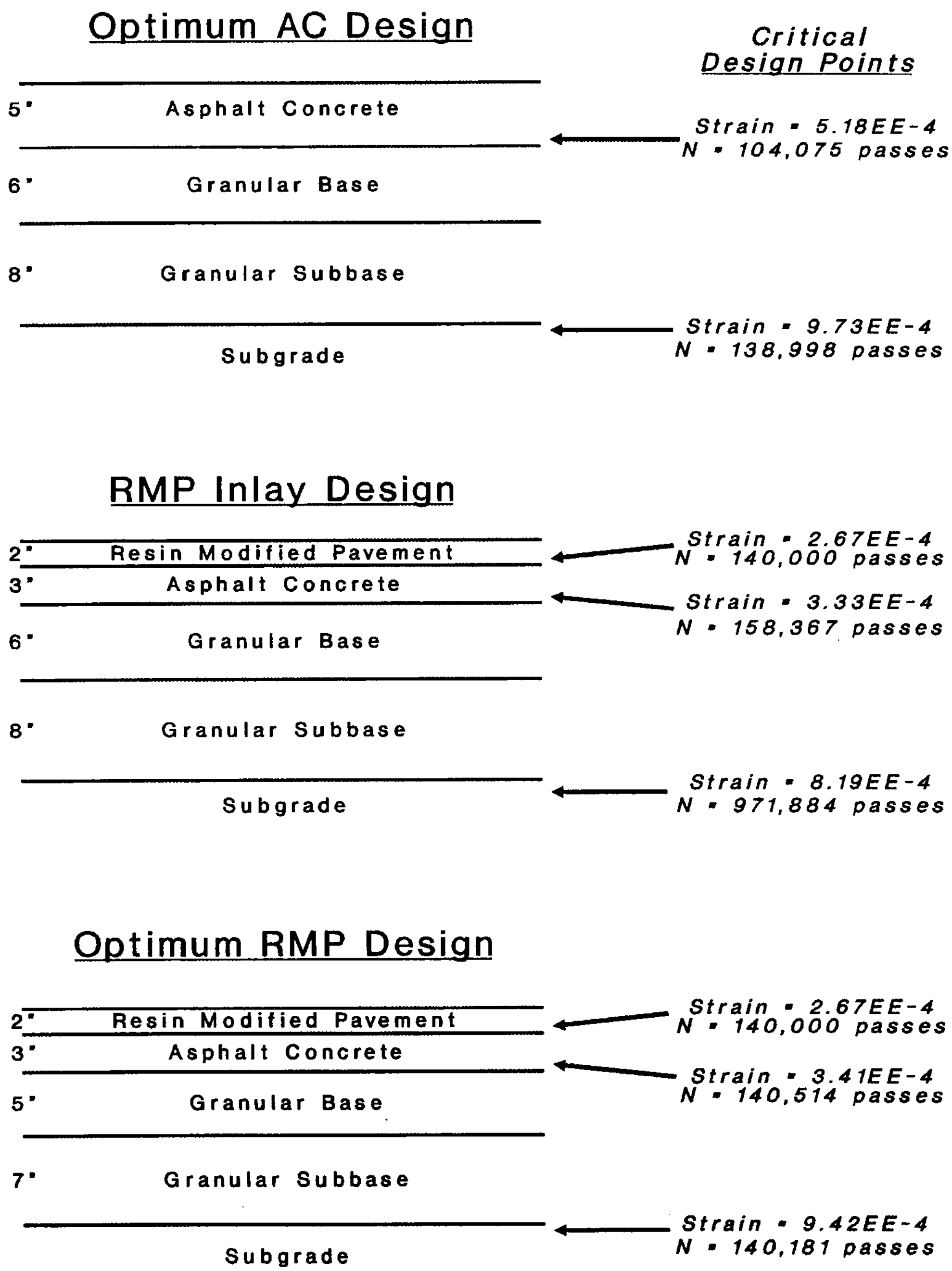

Figure 9.8: Summary of F-16 asphalt concrete and RMP designs

The following list summarizes the important points discovered by this RMP design analysis:

1. The current RMP design approach, involving a one-to-one replacement of AC thickness with RMP thickness, provides more than enough structural capacity to resist fatigue damage in the pavement surfacing and subgrade. In most cases, a marginal amount of conservatism or "over-design" is provided by this design approach.
2. The Corps of Engineers Layered Elastic Design Method is a reasonable approach to conducting a mechanistic design of RMP rehabilitation or new-design projects. The WESPAVE computer program is an easy-to-use tool for the pavement designer to quickly determine critical pavement responses when analyzing airfield designs.
3. Optimum RMP structural designs typically allow for a small to marginal reduction in total pavement thickness, offering an opportunity to save some construction costs when compared to the current RMP design approach.
4. For typical RMP airfield pavement designs, the critical structural point in terms of load-induced fatigue failure appears to be at the bottom of the AC layer. Tensile strains at the bottom of the AC layer are generally increased for RMP designs during warmer seasons, but the critical season for pavement design is when colder temperatures stiffen the AC, making it less fatigue resistant.

5. Typical aircraft loads on suitable RMP designs should not create fatigue cracking in the RMP surfacing itself. Load-induced fatigue cracking in an RMP surfacing should result from fatigue cracking in the AC layer reflecting upwards into the RMP surfacing.
6. Although this RMP structural design analysis does not specifically address a flexible pavement rehabilitation strategy, the analysis results clearly indicate the potential benefits that could be gained by using an RMP overlay instead of an AC overlay for existing flexible pavements. Flexible pavement rehabilitation projects typically occur at some point in the pavement's life when the stiffness of the existing AC surfacing has increased dramatically from its original condition due to environmental aging. The previous RMP design examples show that when the AC material is comparatively stiffer, the typical 50-mm thickness of RMP provides much greater levels of subgrade protection than an equal amount of AC. In these typical flexible pavement rehabilitation conditions, an RMP overlay should provide a significant increase in pavement life when compared to an equivalent thickness of asphalt concrete. A conceptual representation of such an overlay comparison is shown in Figure 9.9. This graph shows how using an RMP overlay instead of a traditional AC overlay would extend the life of the pavement system. Since the RMP overlay reduces the tensile stresses in the underlying original AC by a greater extent than the AC

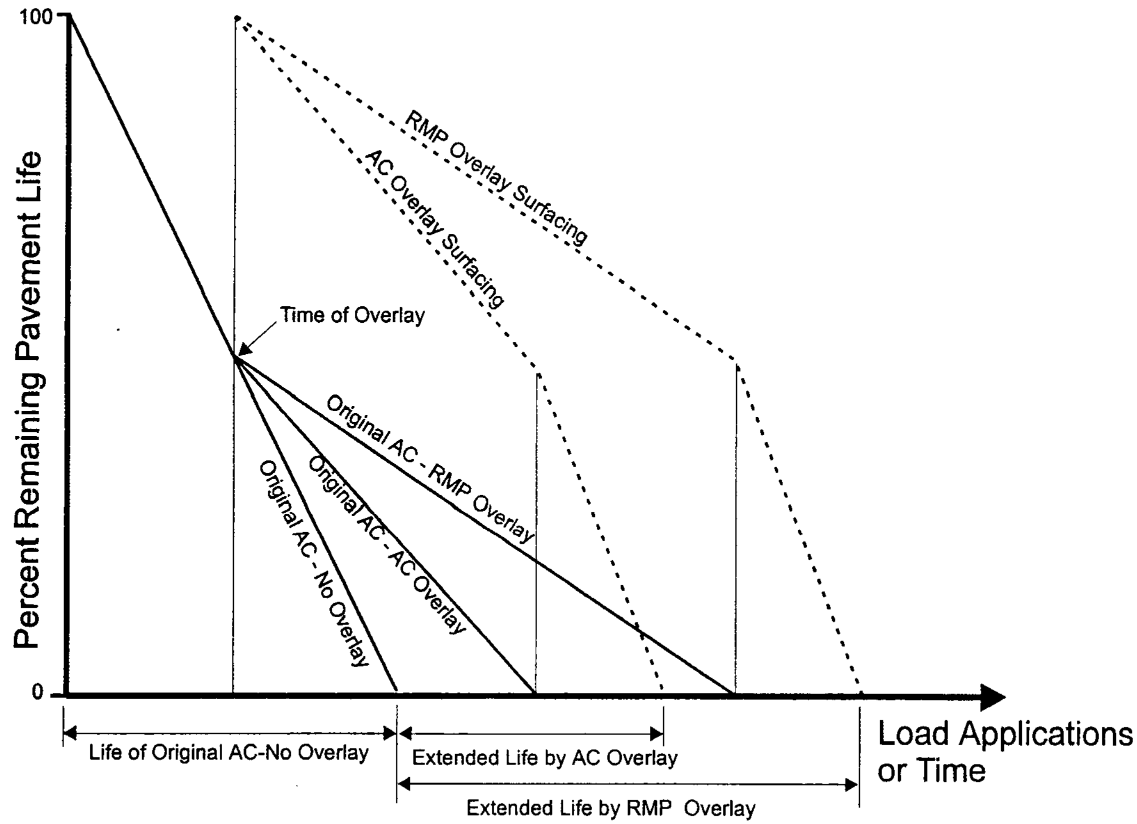

Figure 9.9: Conceptual representation of flexible pavement rehabilitation using asphalt concrete (AC) overlay and RMP overlay

overlay can, the underlying material lasts longer. This delays the onset of reflective cracking from the original AC upwards through the overlay, thus providing a greater extension of pavement life by the RMP overlay.

Chapter 10: Conclusions and Recommendations

CONCLUSIONS

The results of this research study satisfy the primary objectives stated in Chapter 1 of this report, namely:

1. Determine the engineering properties of RMP relating to field performance;
2. Develop a mechanistic pavement design and modeling technique to allow for fundamentally-sound RMP thickness designs and performance predictions.

The significant conclusions reached during the course of this research study which relate to the first objective (engineering properties relevant to field performance) include the following:

1. Documented material research studies on RMP pre-dating the research of this study are few in number and not very comprehensive. These previous studies generally found the RMP strength and stiffness to be between those of typical asphalt concrete (AC) and portland cement concrete (PCC) materials. RMP was noted to be highly resistant to fuel and chemical spills as well as water intrusion because of its relatively low permeability.

2. The only known pavement surfacing material that is comparable to RMP in design and use is Ultra-Thin Whitetopping (UTW). Like RMP, UTW provides a very stiff, rut-resistant surfacing over existing asphalt concrete pavements. Three important differences exist between these two pavement technologies, however: UTW requires relatively tight joint spacing where RMP typically requires no joints; UTW is usually designed for high early strength and next-day traffic where RMP generally requires a 14- to 28-day cure time before traffic is allowed; The unit cost of UTW is approximately twice that of RMP and UTW thicknesses range from 0- to 100-percent more than RMP.
3. Site inspections of past RMP projects indicated several important early performance characteristics: RMP appears to be completely rut-resistant; RMP has little resistance to reflective cracking; Only limited evidence exists of potential thermal cracking for RMP in climates with wide temperature variances; Fatigue cracking seems to be the critical failure mode for an otherwise well-designed and well-constructed RMP project; Early field performance showed that the current empirical structural design approach provides a sufficient thickness design.
4. Strength, stiffness, and fatigue test results indicated a significant visco-elastic component in RMP material behavior (i.e., RMP is somewhat temperature susceptible).

5. RMP has indirect tensile strengths in about the same range as a high-quality AC at lower pavement temperatures and tensile strengths two to three times higher than AC at moderate to high pavement temperatures.
6. RMP generally has 65- to 75-percent less splitting tensile strength when compared to traditional paving-quality PCC.
7. RMP generally has 40- to 60-percent less flexural strength when compared to traditional paving-quality PCC.
8. RMP was measured to have about 10- to 25- percent of the compressive strength of traditional PCC.
9. The resin additive used in the RMP grout appears to have no appreciable effect on the grout's compressive strength.
10. RMP grout appears to have about the same rate of strength gain over time as does standard PCC mortar. Typical RMP grout does seem to achieve most of it's 28-day "ultimate" compressive strength by the 14th day after mixing and placement, indicating that a 14-day or 21- day cure time may be sufficient before trafficking instead of the current 28-day requirement.
11. In terms of material stiffness properties, RMP behaves like a very stiff AC mixture in low to moderate pavement temperatures; but at high pavement temperatures, RMP generally has two to seven times the stiffness of typical AC materials. This relationship goes far in

explaining the RMP's ability to be somewhat flexible and yet remain very resistant to rutting.

12. Poisson's ratio measurements for RMP generally fall between the typical ranges for AC and PCC. A single Poisson's ratio value of 0.27 was determined to be suitable for RMP design and analysis.
13. RMP has thermal coefficients in the same general range as PCC, which is about two to three times lower than that of AC. RMP thermal coefficients appear to be less sensitive to aggregate type when compared to PCC, possibly owing to a damping effect caused by the asphalt cement surrounding and between each aggregate.
14. Freezing and thawing scaling tests indicate that RMP may suffer moderate scaling under severe freezing and thawing conditions. However, previous research which revealed a lower permeability for RMP compared to PCC and field performance to date both indicate that RMP is very resistant to freezing and thawing scaling damage.
15. The fatigue data for various pavement temperatures which were generated by this study appear to be reasonable predictors of RMP fatigue performance.
16. Field measurements of RMP, PCC, and AC skid resistance indicate that under wet conditions, RMP should have about the same skid resistance as typical PCC and AC surfacings. It is predicted that the skid resistance of RMP should gradually improve with time and traffic, similar to AC pavements.

The following list summarizes the significant conclusions relating to this study's objective in developing a mechanistic pavement design and modeling technique for RMP.

1. The Corps of Engineers Layered Elastic Design (COE-LED) Method appears to be a reasonable approach for an RMP mechanistic design and analysis method. The only adjustments required to the current COE-LED method would be to input the appropriate RMP modulus value for the given pavement temperature, use a value of 0.27 for RMP Poisson's ratio, and then compare the computed strain at the bottom of the RMP layer to the appropriate fatigue curve (interpolating between temperature curves when necessary) to determine fatigue life of the RMP material for the given traffic conditions. The computer program WESPAVE was found to be an easy-to-use tool for the pavement designer to quickly determine critical pavement responses when analyzing airfield designs.
2. The current empirical approach used for RMP design was determined to provide a slight to moderate amount of conservatism or "over-design" in most circumstances.
3. Optimum RMP structural designs should allow for a small to marginal reduction in total pavement thickness when compared to the pavement thickness required by the current empirical design approach.
4. For typical RMP airfield pavement designs, the critical structural point in terms of load-induced fatigue failure appears to be at the bottom of

the AC layer, especially during times with colder pavement temperatures.

5. In most circumstances, any load-induced cracking in the surface of an RMP will originate from tensile cracks at the bottom of the AC layer, which will in turn reflect upwards through both the AC and RMP layers.

RECOMMENDATIONS

This research study has demonstrated the practicality and potential usefulness of designing future RMP projects using the Corps of Engineers Layered Elastic Design Method. In many circumstances, the pavement response predictions developed by this proposed design approach were very sensitive to changing input variables of load, temperature, and layer thickness. The immeasurable number of design environments associated with these normal variations points out the importance of using a mechanistic approach to design instead of one driven by limited experience (weak empiricism). It is therefore recommended that the design approach demonstrated in Chapter 9 of this report be used for future airfield RMP design and analysis studies.

It is assumed that the layered elastic design approach used by any agency for flexible pavement design can be modified in a fashion similar to that demonstrated by this study to accommodate RMP designs. This would require a design model capable of handling what is essentially a pavement system with two different surfacing materials bonded together. Since RMP has already been used for several pavement designs involving only vehicular traffic with the potential for a great many more vehicular pavement designs in the future, it is recommended that layered elastic design methods for roads and streets be evaluated for their suitability with RMP design and analysis. Also, flexible pavement rehabilitation with an RMP overlay instead of an asphalt concrete overlay should be considered as a viable option to extend the pavement's service life.

As with any relatively large research effort, there were some questions left unanswered by this study. The following list comprises the most significant recommendations for future research relating to this study:

1. Validation of the RMP design method proposed by this study can only come from trafficking at least several RMP sections and measuring the pavement response. Instrumentation of a new or existing RMP project is recommended to measure true pavement response. Accelerated load testing of several RMP test sections would provide further proof of long-term pavement response and performance.
2. It is recommended that more samples be taken from RMP field sites whenever possible and tested in the laboratory to validate the test data produced in this study. Additional samples of RMP would be especially helpful in further validating the important stiffness and fatigue data used in the proposed RMP design procedure.
3. Field performance of existing RMP sites should be monitored as often as possible to monitor failure modes, track changes in skid resistance, provide real measures of fatigue life, etc. These measures of pavement

performance will provide valuable insight and possible modifications to the mechanistic design and analysis methods for RMP.

4. If any significant changes to the existing RMP material requirements are made in the future, it will be essential to re-evaluate the engineering properties, especially the stiffness and fatigue data used for design and analysis.

Appendix A:

Resin Modified Pavement Mix Design Procedure

RESIN MODIFIED PAVEMENT (RMP) MIX DESIGN PROCEDURE

OPEN-GRADED ASPHALT CONCRETE

Preliminary

Gather representative samples of aggregates and asphalt cement. Sample aggregates according to American Society for Testing and Materials (ASTM) D 75 and asphalt cement according to ASTM D 140. An open-graded asphalt concrete mix design requires a minimum of 45 kg of each aggregate stockpile and 15 L of asphalt cement.

Oven dry aggregate stockpile samples and conduct a sieve analysis (ASTM C 136) on each sample. Determine the combination of aggregate stockpiles that results in a gradation closest to the center of the limiting gradation band specified in CEGS-02746. This stockpile combination will become the blending formula for the open-graded asphalt concrete.

Ensure that the aggregates representing the selected stockpiles and the asphalt cement meet the quality requirements as detailed in CEGS-02746. Measure apparent specific gravity of aggregates (ASTM C 127 and C 128) from each stockpile used in the final gradation. Calculate apparent specific gravity of combined aggregates using the blending formula percentages. Measure specific gravity of asphalt cement (ASTM D 70).

Estimate the optimum asphalt content using the following equation:

$$\text{Optimum asphalt content} = 3.25(\alpha)\Sigma^{0.2}$$

where

α = 2.65/SG

SG = apparent specific gravity of the combined aggregates

Σ = conventional specific surface area = $0.21G + 5.4S + 7.2\,s + 135f$

G = percentage of material retained on 4.75-mm sieve

S = percentage of material passing 4.75-mm sieve and retained on 600-µm sieve

s = percentage of material passing 600-µm sieve and retained on 75-µm sieve

f = percentage of material passing 75-µm sieve

Round the calculated optimum asphalt content value to the nearest tenth of a percent. Use this asphalt content value along with two asphalt contents above this amount and two asphalt contents below this amount in the production of mix design samples. Use 0.5 percent above and below the optimum and 1.0 percent above and below the optimum as the four additional asphalt contents. Calculate maximum theoretical specific gravities for each of these five asphalt cement contents.

Specimen Production

Using the five mix design asphalt contents, produce three 100-mm-diameter Marshall specimens at each asphalt content. Use approximately 800 grams of combined aggregates following the previously determined aggregate blending formula for each specimen. Just before mixing, the temperature of the aggregates should be 145 ± 5 °C and the asphalt cement should be 135 ± 5 °C. With normal mixing procedures, the temperature of the asphalt mixture during compaction is 120 ± 5 °C. Compact the open-graded asphalt concrete specimens with 25 blows from a 4.5-kg Marshall hand hammer on one side of each specimen. Allow the specimens to air cool for a minimum of 4 hours before carefully removing from molds.

Measuring Voids Total Mix (VTM)

Measure the VTM of each open-graded specimen using the following formula:

$$VTM = (1 - WT_{air}/Volume * 1/SG_T) * 100$$

where

WT_{air} = dry weight of specimen

Volume = $\pi/4\ D^2H$

D = diameter H = height

SG_T = maximum theoretical specific gravity

Calculate the average VTM for each of the five asphalt cement contents. Select the optimum asphalt content as that which resulted in a VTM value closest to 30.0 percent. If no VTM averages are in the 30.0 percent range, then slight adjustments to the aggregate gradation may need to be made to achieve the proper void content. Optimum asphalt contents resulting in average VTM values in the 25 to 35 percent range are acceptable, but due to normal production and construction variations, a mix design that provides a 30-percent VTM value is most desirable. (Typical optimum asphalt contents are between 3.5 and 4.5 percent.)

Job-Mix Formula

The open-graded asphalt concrete job-mix formula will consist of the following information:

1. Percentage of each aggregate stockpile.
2. Percentage passing each sieve size for the blended aggregate.
3. Percentage of bitumen.
4. Temperature of discharged asphalt mixture.
5. Voids total mix percentage.

The target temperature of the asphalt mixture when it is discharged from the mixing plant should be 125 ± 5 °C. Select 120 °C when ambient temperatures are relatively high and the haul distance from the asphalt plant to the job site is short. Select 125 °C when either the haul distance is relatively long or the ambient temperatures are relatively cool. Select 130 °C when ambient temperatures are expected to be cool and the haul distance is relatively long. Persistent high winds during construction may also require mix production temperatures to be in the 125- to 130-°C range.

RESIN MODIFIED PORTLAND CEMENT GROUT

Preliminary

Gather representative samples of portland cement, silica sand, Class F fly ash, and resin additive. Minimum sample sizes are 23 kg each of cement, sand, and fly ash, and 4 L of resin additive. Ensure that all materials meet the quality requirements as detailed in CEGS-02746.

Using the grout material proportions specified in CEGS-02746 and shown below, develop a matrix of initial job-mix formulas for laboratory viscosity testing. The goal of the grout mix design is to produce a material formulation which results in a Marsh Flow Cone viscosity of 8.0 to 10.0 seconds. The initial formulations should ensure that a grout formulation can be produced with a Marsh viscosity no greater than the 10.0 seconds maximum. This is accomplished by testing grout formulations with relatively high water/cement (w/c) ratios and the maximum allowable amount of resin additive. Typical initial grout formulations tested in a mix design evaluation are shown below.

	Batch Percentage by Weight			
Material	CEGS-02548 Limits	Trial 1	Trial 2	Trial 3
Portland Cement	34-40	37.0	36.0	35.0
Silica Sand	16-20	18.0	17.8	17.7
Fly Ash	16-20	18.0	17.9	17.8
Water	22-26	24.0	25.0	26.0
Resin Additive	2.5-3.5	3.0	3.3	3.5

Although the grout's w/c ratio is unspecified, the desirable w/c range is 0.65 to 0.75. Lower w/c values are more desirable to reduce the risk of shrinkage cracking and for higher grout strengths. Higher w/c ratios are sometimes necessary to produce grouts with Marsh Flow viscosities less than the 10.0-second maximum value. Therefore, the focus of the initial grout viscosity tests is to determine the minimum w/c ratio that will produce a grout viscosity less than or equal to 10.0 seconds. It is important to remember that the resin additive serves as a plasticizer which reduces grout viscosity while reducing the amount of water required.

The standard laboratory grout batch size should be in the 4,000- to 5,000-g range. Calculate the material batch weights based on the desired proportions. Multiple grout viscosity tests are facilitated by first blending the dry ingredients (cement, sand, fly ash) for each test sample and then adding the appropriate amount of water and resin additive during the mixing process. These dry-ingredient batches should be kept in air-tight containers to prevent loss of material or contamination before mixing. Two replicate samples per blend are appropriate for grout viscosity testing.

Mixing

The equipment needed to effectively mix the resin modified pavement grout includes a laboratory mixer equipped with a wire whip mixing attachment and approximately 10-L-capacity mixing bowl, a calibrated set of weight scales, and various small containers to weigh and transfer mix water and resin additive.

Place dry ingredients into mixing bowl and adjust the bowl height so that the wire whip is just off of or touching the bottom and sides of the bowl. Begin mixing the dry ingredients at a slow speed and immediately add the appropriate amount of water. Once all of the water is added, speed up the mixer to a point where the grout is being thrown onto the sides of the mixing bowl. Mix the grout at this high speed for 5 minutes, then add the appropriate amount of resin additive. Mix the grout again at a high mixing speed for an additional 3 minutes before testing for Marsh Flow viscosity.

Viscosity Testing

The equipment needed to measure grout viscosity includes a Marsh Flow Cone (Figure 1), a 1,000-mL glass or clear plastic graduated cylinder beaker, a 1,500 mL (approximately) empty beaker or bucket, and a stopwatch. Have this equipment set up near the mixing bowl before the end of the 8-minute grout mixing time.

Immediately after mixing the grout, transfer the grout from the mixing bowl to the empty beaker or bucket. Take note of any lumps of material or excess sand in the bottom of the mixing bowl. Excess lumps indicate inadequate mixing and render the grout useless for viscosity testing. Immediately fill the Marsh Flow Cone with about 1,100 mL of grout. A consistent head of grout in the flow cone is achieved for all viscosity tests by marking an 1,100-mL fill line inside the flow cone. The flow cone outlet is plugged by simply placing one's finger over the outlet opening. Immediately after the flow cone is filled to the 1,100-mL fill line, position the cone over the 1,000-mL graduated beaker. Release the grout opening and start the stopwatch timer simultaneously. Measure the time of flow for 1 L of grout from the flow cone to the nearest tenth of a second.

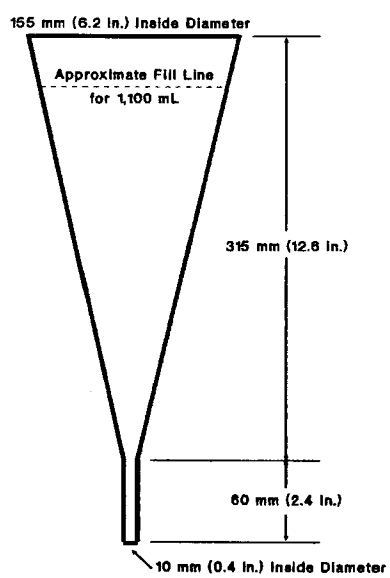

Figure 1: Cross-section of Marsh flow cone

Record each test sample's viscosity, averaging the two replicates for each blend. Adjust the grout mix proportions as needed with the following considerations:

1. Any grout viscosity between 8.0 and 10.0 seconds is acceptable. It should be noted, however, that when field construction temperatures are expected to be comparatively high (greater than 32 °C) and/or the open-graded asphalt concrete voids are expected to be considerably low (less than 30 percent), then lower viscosity grouts will help to ensure easy grout application and full grout penetration. In most cases, these variables are unknown; therefore, it is prudent to select the grout formulation which has the lowest viscosity.

2. It is best to develop a grout job-mix formula with water and resin additive contents below the maximum allowable limits to allow for small additions of these ingredients in the field if necessary to meet viscosity requirements.

3. Lower w/c ratios are more desirable for a number of reasons: they tend to produce grouts of higher strengths; they reduce the chances for drying shrinkage cracking; they produce grouts which are more consistent and better able to keep the sand in suspension during mixing and placement.

4. When the sand is noted to settle out of solution during or immediately after mixing, it can be expected that similar problems would occur in the field during construction. This problem can be remedied by reducing the amount of sand and increasing the amount of fly ash (both within the specified tolerances) to produce a slightly creamier grout.

5. When it becomes impossible to meet the viscosity requirements within the specified limits for material quantities, there usually is a problem with a particular ingredient. Some of these deficiencies are detectable, while others are not. These material deficiencies may include one or more of the following: grout sand which is too coarse, portland cement which is highly reactive during the early stages of the hydration process, fly ash with excess cementitious nature. When it is possible to isolate the problem material in these instances, the only recourse is to substitute another material from another source whose physical or chemical difference will likely solve the problem.

Job-Mix Formula

The grout job-mix formula will consist of the following information:

1. Percentage (by weight) of each mixture ingredient rounded to the nearest tenth of a percent.

2. Type and source of portland cement.

3. Source of fly ash, silica sand, and resin additive.

4. Marsh Flow Cone viscosity of job-mix-formula grout.

Appendix B:

Resin Modified Pavement Guide Specification

DEPARTMENT OF THE ARMY CEGS–02746 (February 1997)
U.S. ARMY CORPS OF ENGINEERS

GUIDE SPECIFICATION FOR MILITARY CONSTRUCTION

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SECTION 02746

RESIN MODIFIED PAVEMENT

02/97

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NOTE: This guide specification covers the requirements for resin modified pavement surfacing material. This guide specification is to be used in the preparation of project specifications in accordance with ER 1110–345–720.

****

PART 1 GENERAL

****

NOTE: See Additional Notes A and B.

****

1.1 REFERENCES

****

NOTE: Issue (date) of references included in project specifications need not be more current than provided by the latest change (Notice) to this guide specification.

****

The publications listed below form a part of this specification to the extent referenced. The publications are referred to in the text by basic designation only.

AMERICAN SOCIETY FOR TESTING AND MATERIALS (ASTM)

\–ASTM C 88–\	(1990) Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate
\–ASTM C 131–\	(1989) Resistance to Degradation of Small–Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine
\–ASTM C 136–\	(1995) Sieve Analysis of Fine and Coarse Aggregates
\–ASTM C 150–\	(1995) Portland Cement
\–ASTM C 618–\	(1994) Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete
\–ASTM D 75–\	(1992) Sampling Aggregates
\–ASTM D 140–\	(1993) Sampling Bituminous Materials
\–ASTM D 2216–\	(1992) Laboratory Determination of Water (Moisture) Content of Soil and Rock
\–ASTM D 3381–\	(1992) Viscosity–Graded Asphalt Cement for Use in Pavement Construction
\–ASTM D 4791–\	(1989) Flat or Elongated Particles in Course Aggregate

CORPS OF ENGINEERS (COE)

\–COE CRD–C 300–\	(1990) Specifications for Membrane-Forming Compounds for Curing Concrete

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1.2 SUBMITTALS

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NOTE: Submittals must be limited to those necessary for adequate quality control. The importance of an item in the project should be one of the primary factors in determining if a submittal for the item should be required.

Indicate submittal classification in the blank space using "GA" when the submittal requires Government approval or "FIO" when the submittal is for information only.

****

Government approval is required for submittals with a "GA" designation; submittals having an "FIO" designation are for information only. The following shall be submitted in accordance with Section \=01300=\ SUBMITTAL PROCEDURES:

SD-09 Reports\

Coarse and Fine Aggregate\; *GA*\. *Open-graded Mix Aggregate Gradation*\; *GA*\. *Bituminous Material*\; *GA*\. *Slurry Grout Sand*\; *GA*\. *Fly Ash*\; *GA*\. *Slurry Grout Formula*\; *GA*\. Copies of test results. Slurry grout viscosity tests shall be conducted immediately prior to application on the pavement surface and 30 minutes thereafter.

SD–13 Certificates\

Cement\; *GA*\. *Cross Polymer Resin*\; *GA*\. *Curing Compound*\; *GA*\. Copies of certificates.

SD–14 Samples\

Open-graded Mix\; *[_____]*\. *Slurry Grout Job–Mix–Formula*\; *[_____]*\. Materials required to produce the open-graded mixture and slurry grout job–mix–formulas shall be submitted in the quantities indicated below.

Aggregates representing each stockpile to be used in the production of the open-graded mixture \^45 kg^\ \~100 pounds~\ each

Bituminous Material \^19 liters^\ \~5 gallons~\

Slurry Grout Sand \^23 kg^\ \~50 pounds~\

Fly Ash \^23 kg^\ \~50 pounds~\

Cement \^23 kg^\ \~50 pounds~\

Cross Polymer Resin \^4 liters^\ \~1 gallon~\

Samples shall be delivered, along with the Contractor's preliminary job mix formulas, 30 days before starting production to U.S. Army Engineer Waterways Experiment Station, 3909 Halls Ferry Road, Vicksburg, Mississippi, 39180–6199, ATTN: CEWES–GP–Q.

1.3 PLANT, EQUIPMENT, MACHINES, AND TOOLS

The bituminous plant shall be of such capacity as to produce the quantities of bituminous mixtures required for the project. Hauling equipment, paving machines, rollers, miscellaneous equipment, and tools shall be provided in sufficient numbers and capacity and in proper working condition to place the bituminous paving mixtures at a rate equal to the plant output. The additional requirements for construction of the Resin Modified Pavement (RMP) are a concrete batch plant, a ready mix truck or portable mixer for grout mixing, and a small \^2.7 metric ton (3–ton)^\ \~3–ton~\ tandem steel wheeled vibratory roller for compaction.

1.4 SAMPLING AND TESTING

1.4.1 Aggregates

1.4.1.1 General

\~ASTM D 75~\ shall be used in sampling coarse and fine aggregates. Points of sampling will be designated by the Contracting Officer. All tests necessary to determine compliance with the specified requirements shall be made by the Contractor.

1.4.1.2 Sources

Sources of aggregates shall be selected well in advance of the time that the materials are required in the work. Samples shall be submitted 30 days before starting production. If a sample of material fails to meet specification requirements, the material represented by the sample shall be replaced, and the cost of testing the replaced sample shall be at the expense of the Contractor. Approval of the source of the aggregate does not relieve the Contractor of the responsibility to deliver aggregates that meet the specified requirements.

1.4.2 Bituminous Materials

Samples of bituminous materials shall be obtained in accordance with \–ASTM D 140–\. Sources shall be selected in advance of the time materials will be required for the work. In addition to the initial qualification testing of bituminous materials, samples shall be obtained and tested before and during construction when shipments of bituminous materials are received, or when necessary to assure that some condition of handling or storage has not been detrimental to the bituminous material.

1.5 DELIVERY, STORAGE, AND HANDLING OF MATERIALS

1.5.1 Mineral Aggregates

Mineral aggregates shall be delivered to the site of the bituminous mixing plant and stockpiled in such a manner as to preclude segregation or contamination with objectionable material.

1.5.2 Bituminous Materials

Bituminous materials shall be maintained below a temperature of \^150 degrees C^\ \~300 degrees F~\ during storage and shall not be heated by the application of a direct flame to the walls of storage tanks or transfer lines. Storage tanks, transfer lines and weigh buckets shall be thoroughly cleaned before a different type or grade of bitumen is introduced into the system.

1.5.3 Slurry Grout Sand

Slurry grout sand shall be stored at the grout production site so as to prevent contamination with foreign materials and saturation with rain water. Moisture content of this sand shall be determined just prior to grout production so that corrections to the job mix formula water content can be made to compensate for any moisture in the sand.

1.6 ACCESS TO PLANT AND EQUIPMENT

The Contracting Officer shall have access at all times to all parts of the bituminous plant for checking adequacy of any equipment in use; inspecting operation of the plant; verifying weights, proportions, and character of materials; and checking temperatures maintained in preparation of the mixtures.

PART 2 PRODUCTS

2.1 AGGREGATE

Aggregate shall consist of crushed stone, or crushed gravel without sand or other inert finely divided mineral aggregate. The portion of materials retained on the \^4.75 mm^\ \~No. 4~\ sieve shall be known as coarse aggregate, the portion passing the \^4.75 mm^\ \~No. 4~\ sieve and retained on the \^0.075 mm^\ \~No. 200~\ sieve as fine aggregate. Sieve analysis of coarse and fine aggregates shall be conducted in accordance with \–ASTM C 136–\.

2.1.1 Coarse Aggregate

Coarse aggregate shall consist of sound, tough, durable particles, free from adherent films of matter that would prevent thorough coating with the bituminous material. The percentage of wear shall not be greater than 40 percent when tested in accordance with \–ASTM C 131–\. The sodium sulfate soundness loss shall not exceed 9 percent, after five cycles, when tested in accordance with \–ASTM C 88–\. Aggregate shall contain at least 70 percent by weight of crushed pieces having two or more fractured faces. The area of each fractured face shall be equal to at least 75 percent of the smallest mid–sectional area of the piece. When two fractured faces are contiguous, the angle between the planes of fractures shall be at least 30 degrees to count as two fractured faces. Fractured faces shall be obtained by artificial crushing.

2.1.2 Crushed Aggregates

Particle shape of crushed aggregates shall be essentially cubical. Quantity of flat and elongated particles in any sieve size shall not exceed 8 percent by weight, when determined in accordance with \-ASTM D 4791-\.

2.1.3 Open-graded Mix Aggregate

The gradations in Table I represent the limits which shall determine the suitability of open-graded mix aggregate for use from the sources of supply. The aggregate, as finally selected, shall have a gradation within the limits designated in Table I and shall not vary from the low limit on one sieve to the high limit on the adjacent sieve, or vice versa, but shall be uniformly graded from coarse to fine.

TABLE I
OPEN-GRADED MIX AGGREGATE

Sieve Size	Percent by Weight Passing
3/4 in.	100
1/2 in.	54–76
3/8 in.	38–60
No. 4	10–26
No. 8	8–16
No. 30	4–10
No. 200	1–3

Table I is based on aggregates of uniform specific gravity; the percent passing various sieves may be changed by the Contracting Officer when aggregates of varying specific gravities are used. Adjustments of percentages passing various sieves may be directed by the Contracting Officer when aggregates vary more than 0.2 in specific gravity.

2.1.4 Slurry Grout Sand

Slurry grout sand shall consist of clean, sound, durable, particles of processed silica sand that meets the requirements for wear and soundness specified for coarse aggregate. The sand shall contain no clay, silt, or other objectionable matter. The gradations in Table II represent the limits which shall determine the suitability of silica sand for use from the sources of supply.

TABLE II
FINE SAND FOR SLURRY GROUT

Sieve Size	Percentage by Weight Passing
No. 16	100
No. 30	95–100
No. 200	0–2

The sand gradations shown are based on sand of uniform specific gravity, and the percentages passing the various sieves will be subject to appropriate correction by the Contracting Officer when aggregates of varying specific gravities are used.

2.1.5 Filler

If filler in addition to that naturally present in the aggregate is necessary, it shall be fly ash. Fly ash shall have at least 95 percent by weight of material passing the \^0.075 mm^\ \~No. 200~\ sieve. Fly ash shall conform to \–ASTM C 618–\ Class F requirements.

2.2 BITUMINOUS MATERIAL

Bituminous material shall conform to the requirements of \–ASTM D 3381–\ and shall be of the viscosity grade [AC-10] [AC–20] [AC–30] [AR–4000] [AR–8000] with an original penetration of 40 to 100.

2.3 CEMENT

The cement used in the slurry grout shall be portland cement conforming to \–ASTM C 150–\, Type [I] [II] [III] [V].

2.4 CROSS POLYMER RESIN

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NOTE: See Additional Note C.

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NOTE: A complete description of the Marsh flow cone and the grout viscosity test method is found in the Engineer Technical Letter ETL 1110-1-177 "Use of Resin Modified Pavement (RMP)."

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A cross polymer resin of styrene and butadiene, Prosalvia L7, shall be utilized as a plasticizing and strength producing agent. After mixing the resin into the slurry grout, the mixture shall have a viscosity which would allow it to flow from a Marsh Cone in accordance with Table III. A Marsh cone has dimensions of 155 mm base inside diameter, tapering 315 mm to a tip inside diameter of 10 mm. The 10 mm diameter neck shall have a length of 60 mm.

TABLE III
SLURRY GROUT VISCOSITY

Time Elapsed After Addition of PL7	Marsh Flow Cone Viscosity
0 to 30 minutes	8 to 10 seconds
After 30 minutes	9 to 11 seconds

2.5 CURING COMPOUND

Membrane–forming curing compound shall be white pigmented compounds conforming to \–COE CRD–C 300–\.

2.6 JOB MIX FORMULA AND COMPOSITION OF SLURRY GROUT

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NOTE: See Additional Note D.

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Note: A complete description of the proper methods used to produce job mix formulas for the open-graded bituminous mixture and slurry grout is found in the Engineer Technical Letter ETL 1110-1-177 "Use of Resin Modified Pavement (RMP)."

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2.6.1 Job Mix Formula

The Job Mix Formula (JMF) for the open-graded bituminous mixture shall be furnished by the Contractor and approved by the Government. No payment will be made for mixtures produced prior to the approval of the JMF by the Contracting Officer. The JMF will indicate the percentage of each stockpile, the percentage passing each sieve size, the percentage of bitumen, and the temperature of the completed mixture when discharged from the mixer. The tolerances given in Table IV for sieve analysis, bitumen content, and temperature shall be applied to quality control test results on the open-graded bituminous mixture as discharged from the mixing plant.

TABLE IV
JOB–MIX–FORMULA TOLERANCES

Material	Tolerance Plus or Minus
Aggregate passing No. 4 or larger sieves	4 percent
Aggregate passing Nos. 8 and 30 sieves	3 percent
Aggregate passing No. 200 sieve	1 percent
Bitumen	0.20 percent
Temperature of discharged mix	\^11°C^\ \~20°F~\

2.6.2 Composition of Slurry Grout

The Job Mix Formula (JMF) for the slurry grout shall be furnished by the Contractor and approved by the Government. The slurry grout job mix formula shall be developed using the proportions given in Table V.

TABLE V
RESIN MODIFIED CEMENT SLURRY GROUT MIXTURE PROPORTIONS

Material	Percent by Weight
Silica Sand	16–20
Fly Ash	16–20
Water	22–26
Type I Cement	34–40
Cross Polymer Resin	2.5–3.5

Approximately \^12 kg to 15 kg^\ \~22 pounds to 28 pounds~\ of mixed slurry grout will fill in one square \^meter^\ \~yar~\ (\^25 mm^\ \~1 inch~\ thickness) of open-graded bituminous mixture with 25 to 35 percent voids total mix.

PART 3 EXECUTION

3.1 WEATHER LIMITATIONS

The bituminous mixture shall not be placed upon a wet surface, in rain, or when the surface temperature of the underlying course is less than \^10 degrees C.^\ \~50 degrees F.~\ The temperature requirements may be waived by the Contracting Officer. Once the bituminous mixture has been placed and if rain is imminent, protective materials, consisting of rolled polyethylene sheeting at least \^0.1 mm (4 mils)^\ \~4 mils~\ thick of sufficient length and width to cover the mixture shall be placed. If the open-graded bituminous mixture becomes saturated, the Contractor shall allow the pavement voids to thoroughly dry out prior to applying the slurry grout.

3.2 PREPARATION OF OPEN-GRADED MIXTURES

Rates of feed of aggregates shall be regulated so that moisture content and temperature of aggregates will be within tolerances specified. Aggregates and bitumen shall be conveyed into the mixer in proportionate quantities required to meet the JMF. Mixing time shall be as required to obtain a uniform coating of the aggregate with the bituminous material. Temperature of bitumen at time of mixing shall not exceed \^135 degrees C.^\ \~275 degrees F.~\ Temperature of aggregate in the mixer shall not exceed \^150 degrees C^\ \~300 degrees F~\ when bitumen is added. Overheated and carbonized mixtures or mixtures that foam shall not be used.

3.3 WATER CONTENT OF AGGREGATES

Drying operations shall reduce the water content of mixture to less than 0.75 percent. Water content shall be determined in accordance with \~ASTM D 2216~\; weight of sample shall be at least \^500 grams.^\ \~500 grams.~\ The water content shall be reported as a percentage of the total mixture.

3.4 STORAGE OF MIXTURE

The open-graded bituminous mixture shall not be stored for longer than one hour prior to hauling to the job site.

3.5 TRANSPORTATION OF MIXTURE

Transportation from the mixing plant to the job site shall be in trucks having tight, clean, smooth beds lightly coated with an approved releasing agent to prevent adhesion of mixture to truck bodies. Diesel fuel shall not be used as a releasing agent. Excessive release agent shall be drained prior to loading. Each load shall be covered with canvas or other approved material of ample size to protect mixture from the weather and to prevent loss of heat. Loads that have crusts of cold, unworkable material or have become wet will be rejected. Hauling over freshly placed material will not be permitted.

3.6 TEST SECTION

Prior to full production, and in the presence of the Contracting Officer, the Contractor shall prepare and place a quantity of open-graded bituminous mixture and slurry grout according to the JMF. The test section shall be a minimum of \^30 meters^\ \~100 feet~\ long and \^6 meters^\ \~20 feet~\ wide placed in one section and shall be of the same depth specified for the construction of the course which it represents. The equipment used in construction of the test section shall be the same type and weight to be used on the remainder of the course represented by the test section. The test section shall meet the requirements specified in paragraph ACCEPTABILITY OF WORK. If the test section should fail to meet these requirements, the necessary adjustments to the mix design, plant operation, and/or construction procedures shall be made. Additional test sections, as required, shall be constructed and evaluated for conformance to the specifications at the expense of the Contractor.

3.7 SURFACE PREPARATION OF UNDERLYING COURSE

Prior to placing of open-graded bituminous mixture, the underlying course shall be cleaned of all foreign or objectionable matter with power brooms and hand brooms.

3.8 TACK COATING

Contact surfaces of previously constructed pavement shall be sprayed with a coat of bituminous material as specified in Section \=02558=\ BITUMINOUS TACK COAT.

3.9 PLACING OPEN-GRADED BITUMINOUS MIXTURE

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NOTE: The amount of rolling required to achieve the required voids total mix criteria is usually 1 to 3 passes of the 2.7 metric ton (3–ton) tandem steel wheel roller in the static mode. The appropriate temperature of the freshly placed bituminous mixture required to prevent undue shoving and cutting from the roller is usually in the 50 to 70 degrees C (120 to 160 degrees F) range. The actual number of required passes and temperature range for rolling should be determined during construction and subsequent evaluation of the test section.

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The mix shall be placed at a temperature of not less than \^80 degrees C^\ \~175 degrees F~\. Upon arrival, the mixture shall be spread to the full width (minimum \^3 meters^\ \~10 feet~\) by an approved bituminous paver. It shall be struck off in a uniform layer of such depth that, when the work is completed, it shall have the required thickness indicated. The speed of the paver shall be regulated to eliminate pulling and tearing of the bituminous mat. Unless otherwise directed, placement of the mixture shall begin along the center line of a crowned pavement or along the highest side of a sloped cross–section. The mixture shall be placed in consecutive adjacent strips. On areas where irregularities or unavoidable obstacles make the use of mechanical spreading and finishing equipment impractical, the mixture may be spread, raked, and luted by hand tools.

3.9.1 Rollers

Small (\^2.7 metric ton^\ \~3–ton~\ maximum) tandem steel wheel vibratory rollers shall be used to smooth over the surface of freshly placed open-graded bituminous mixture. The vibratory unit shall be turned off during smoothing of the bituminous mixture. Roller shall be in good condition, capable of operating at slow speeds to avoid displacement of the bituminous mixture. The number, type, and weight of rollers shall be sufficient to roll the mixture to the voids total mix requirement of 25 to 35 percent while it is still in a workable condition. The use of equipment which causes excessive crushing of the aggregate will not be permitted.

3.9.2 Smoothing of Open-Graded Bituminous Mixture

The open-graded bituminous mixture shall be smoothed with one to three passes of the prescribed roller without vibration. The temperature of the freshly placed open-graded bituminous mixture shall be low enough to prevent excessive shoving or cutting of the mat under the roller.

3.9.3 Protection of Ungrouted Pavement

The Contractor shall protect the ungrouted pavement and its appurtenances against contamination from mud, dirt, wind blown debris, waterborne material, or any other contamination which could enter the void spaces of the open-graded bituminous mixture before grout application. Protection against contamination shall be accomplished by keeping the construction site clean and free of such contaminants and by covering the ungrouted pavement with protective materials when directed by the Contracting Officer. Such protective materials shall consist of rolled polyethylene sheeting as described in paragraph WEATHER LIMITATIONS. The sheeting may be mounted on either the paver or a separate movable bridge from which it can be unrolled without dragging over the pavement surface.

3.10 PREPARATION OF SLURRY GROUT

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NOTE: Generally, the cross polymer resin should be added to the grout mixture at the batch plant if the haul distance is less than 20 minutes. If the haul distance is greater than 20 minutes, the cross polymer resin should be added to the grout mixture at the job site.

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The slurry grout shall be mixed using a batch plant, portable mixer and/or ready–mix truck and according to mix proportions stated in the approved JMF. The cross polymer resin shall be added to the mixture after all other ingredients have been thoroughly mixed. When using ready–mix trucks for transporting slurry grout, the grout mixture shall be thoroughly mixed at the job site immediately before application for a minimum of 10 minutes. Thorough mixing shall be accomplished by rotating the mixing drum at the maximum allowable revolutions per minute.

3.11 PLACING SLURRY GROUT

Temperature of the bituminous mixture shall be less than \^38 degrees C^\ \~100 degrees F~\ before applying grout. Each batch of slurry grout shall be tested at the job site immediately before placement and shall be used in the finished product only if it meets the requirements specified in paragraph ACCEPTABILITY OF WORK. The slurry grout shall be spread over the bituminous mixture using a spreader or squeegees. The application of the slurry grout shall be sufficient to fill the internal voids of the open-graded bituminous mixture. The grouting operation shall begin at the lowest side of the sloped cross–section and proceed from the low side to the high side. The practical limit for the surface slope of an RMP section is 2 percent. Pavement slopes up to 5 percent can be constructed, but excess hand work and grout overruns are to be expected at slopes greater than 2 percent. The slurry grout shall be placed in successive paving lanes with a maximum width of \^6 meters.^\ \~20 feet.~\ The use of \^50 by 100 mm (2–inch by 4–inch)^\ \~2–inch by 4–inch~\ strips of lumber as wooden battens separating each of the grouting lanes and the RMP from adjacent pavements is optional. The direction of the grouting operation shall be the same as used to pave the open-graded bituminous mixture. The small (\^2.7 metric ton (3–ton)^\ \~3–ton~\ maximum) tandem steel wheel roller (vibratory mode) passing over the grout covered bituminous mixture shall be used to promote full penetration of the slurry grout into the void spaces.

3.12 JOINTS

3.12.1 Joints Between Successive Lanes of RMP

Joints between successive lanes of RMP shall be made in such a manner as to ensure a continuous bond between the paving lanes. All RMP joints shall have the same texture, density, and smoothness as other sections of the course.

3.12.2 Joints Between RMP and Adjacent Pavements

Joints between the RMP and any surrounding pavement surfaced with portland cement concrete shall be saw cut to the full thickness of the RMP layer and filled with a joint sealant material approved by the Contracting Officer.

3.13 CURING

The curing compound shall be applied to the finished pavement surface within 2 hours of the completed slurry grout application. The curing compound shall be applied by means of an approved pressurized spraying machine. Application of the curing compound shall be made in one or two coats with a total application rate of not more than \^10 square meters per liter.^\ \~400 square feet per gallon.~\

3.14 PROTECTION OF GROUTED PAVEMENT

The Contractor shall protect the pavement and its appurtenances against both public traffic and traffic caused by the Contractor's employees and agents for a period of 28 days. Any damage to the pavement occurring prior to final acceptance shall be repaired or the pavement replaced at the Contractor's expense. In order that the pavement be properly protected against the effects of rain before the pavement is sufficiently hardened, the Contractor will be required to have available at all times materials for the protection of the edges and surfaces of the unhardened RMP. The protective materials and method of application shall be the same as previously described in paragraph WEATHER LIMITATIONS. When rain appears imminent, all paving operations shall stop, and all available personnel shall begin covering the surface of the hardened RMP with protective covering.

3.15 ACCEPTABILITY OF WORK

3.15.1 General

Routine testing for acceptability of work shall be performed by the Contractor and approved by the Contracting Officer. Additional tests required to determine acceptability of non–conforming material shall be performed by the Contractor at the expense of the Contractor. When a section of pavement fails to meet the specification requirements, that section shall be totally removed and replaced at the Contractor's expense. The Contracting Officer reserves the right to sample and test any area which appears to deviate from the specification requirements.

3.15.2 Field Sampling of RMP Materials

3.15.2.1 Open-Graded Bituminous Mixture

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NOTE: Voids total mix of laboratory specimens and ungrouted field cores shall be calculated using the following formula:

$$VTM = (1 - WT_{air}/Volume \times 1/SG_T) \times 100$$

where

VTM = voids total mix
WT_{air} = dry weight of specimen
$Volume = \pi/4\ D^2H$
D = diameter H = height
SG_T = theoretical specific gravity

****

Samples of open-graded bituminous mixture shall be taken from loaded trucks for every 1,000 square \^meters^\ \~yards~\ of pavement, but not less than two samples for each day of paving for determining asphalt content, aggregate gradation, and laboratory compacted voids total mix. Laboratory specimens of open-graded bituminous material shall be compacted in \^101.6 mm (4 inch)^\ \~4 inch~\ diameter molds to a \^50.8 mm (2 inch)^\ \~2 inch~\ thickness using 25 blows on one side from a Marshall hand hammer. Test results from the sampled open-graded bituminous mixture shall be compared to the approved job--mix--formula and approved by the Contracting Officer for acceptance.

3.15.2.2 Slurry Grout

Each batch of slurry grout shall be tested for viscosity at the job site after thorough mixing and before application. Any batch of slurry grout failing to meet the viscosity specified requirements shall be rejected and removed from the job site. Slurry grout with visible amounts of sand settling out of suspension during application shall be rejected and removed from the job site.

3.15.2.3 Core Samples

Random core samples shall be taken from the in--place open-graded bituminous mixture before and after application of the slurry grout. The Contractor shall take at least two field core samples before grout application and two after grout application for every 1,000 square \^meters^\ \~yards~\ of finished RMP. Half of the core samples taken after grout application shall be taken from joints between successive grouting lanes. Field core samples shall be \^101.6 or 152.4 mm (4 or 6 inch)^\ \~4 or 6 inch~\ diameter and extend the full depth of the RMP surface layer. The ungrouted core samples shall be tested for thickness. The grouted core samples shall be visually inspected for acceptable grout penetration. Acceptable grout penetration shall be through the full thickness of the RMP layer with a minimum of 90 percent of the visible void spaces filled with slurry grout. After testing, the Contractor shall turn over all cores to the Contracting Officer. Core holes in ungrouted RMP shall be filled with hot open-graded bituminous material and leveled to match the surrounding pavement surface. Core holes in grouted RMP shall be filled within 24 hours from the time of coring with RMP material, low-shrinkage portland cement concrete material, or other approved patching material.

3.15.3 Thickness and Surface--Smoothness Requirements

Finished surface of RMP, when tested as specified below, shall conform to the thickness specified and to surface smoothness requirements specified in Table VI.

TABLE VI
SURFACE–SMOOTHNESS TOLERANCES

Direction of Testing	Resin Modified Pavement Tolerance
Longitudinal	\^6 mm^\ \~1/4 inch~\
Transverse	\^6 mm^\ \~1/4 inch~\

3.15.3.1 Thickness

The thickness of the RMP shall meet the requirements shown on the contract drawings. The measured thickness of the RMP shall not exceed the design thickness by more than \^13 mm,^\ \~1/2 inch,~\ or be deficient in thickness by more than \^6 mm.^\ \~1/4 inch.~\

3.15.3.2 Surface Smoothness

Finished surfaces shall not deviate from testing edge of a \^3.66 meter (12 foot)^\ \~12–foot~\ straightedge more than the tolerances shown for the respective pavement category in Table VI.

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ADDITIONAL NOTES

NOTE A: For additional information on the use of all CEGS, see CEGS–01000 CEGS GENERAL NOTES.

NOTE B: A representative of the Airfield and Pavements Division, Geotechnical Laboratory, U.S. Army Engineer Waterways Experiment Station (WES) should be consulted in the planning and designing of an RMP.

NOTE C: The cross polymer resin to be used in the slurry grout, Prosalvia-7, is available from the Alyan Corporation, P.O. Box 788, Vienna, VA 22183, (703) 573-8134.

NOTE D: It is recommended that the job mix formula for the open-graded bituminous mixture and the mixture proportions for the slurry grout be approved by the appropriate WES representative. On a case by case basis, this approval may result from a simple review of the Contractor's mix design test reports, or it may require certification of the mix design by repeating some or all of the required mix design tests. This recommendation is to ensure that proper laboratory procedures are used to determine mix designs for this new paving process.

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— End of Section —

Appendix C:

Strength and Stiffness Test Results

Table C.1: RMP Indirect Tensile Strength Test Results			
Sample Source	Test Temperature (C)	Tensile Strength (kPa)	Statistics
		2789	μ = 2525 kPa
Lab	5	2223	s = 285 kPa
		2563	v = 11.3%
		1599	μ = 1560 kPa
Lab	25	1550	s = 35 kPa
		1532	v = 2.2%
		483	μ = 571 kPa
Lab	40	699	s = 113 kPa
		531	v = 19.9%
		1784	μ = 2097 kPa
Altus	5	2463	s = 343 kPa
		2045	v = 16.3%
		1967	μ = 1760 kPa
Altus	25	1648	s = 180 kPa
		1664	v = 10.2%
		440	μ = 590 kPa
Altus	40	633	s = 134 kPa
		697	v = 22.7%
		2007	μ = 2085 kPa
McChord	5	2044	s = 105 kPa
		2204	v = 5.0%
		1445	μ = 1613 kPa
McChord	25	1711	s = 146 kPa
		1682	v = 9.0%
		658	μ = 816 kPa
McChord	40	819	s = 157 kPa
		971	v = 19.2%

Table C.2: RMP Grout Cube Compressive Strength Test Results			
Sample Source	Age (days)	Compressive Strength (MPa)	Statistics
		12.7	μ = 12.8 MPa
APG	7	12.2	s = 0.7 MPa
		13.6	v = 5.5%
		20.1	μ = 17.4 MPa
APG	14	16.2	s = 2.4 MPa
		15.8	v = 13.7%
		20.0	μ = 23.6 MPa
APG	28	24.8	s = 3.2 MPa
		26.0	v = 13.5%
		16.8	μ = 15.5 MPa
McChord	7	14.6	s = 3.2 MPa
		15.2	v = 13.5%
		21.0	μ = 19.3 MPa
McChord	14	18.0	s = 1.5 MPa
		18.9	v = 8.0%
		20.0	μ = 21.3 MPa
McChord	28	21.5	s = 1.2 MPa
		22.3	v = 5.5%
		16.8	μ = 17.1 MPa
Johnstown	7	16.6	s = 0.6 MPa
		17.8	v = 3.8%
		22.0	μ = 20.4 MPa
Johnstown	14	18.7	s = 1.7 MPa
		20.4	v = 8.1%
		22.0	μ = 22.1 MPa
Johnstown	28	22.7	s = 0.6MPa
		21.6	v = 2.5%

Table C.3: Resilient Modulus Test Results for Laboratory RMP Samples				
Test Temperature (°C)	Modulus (MPa)	Statistics	Poisson's Ratio	Statistics
	20641		0.164	
	21576		0.144	
	17159		0.238	
	17367	$\mu = 19{,}168$	0.201	$\mu = 0.195$
5	17535	s = 1921	0.240	s = 0.043
	20914	v = 10.0%	0.139	v = 21.9%
	17249		0.266	
	21154		0.168	
	20558		0.187	
	17524		0.198	
	12402		0.283	
	12077		0.255	
	10318		0.291	
	12064	$\mu = 11{,}213$	0.316	$\mu = 0.258$
25	10459	s = 1049	0.240	s = 0.039
	9775	v = 9.4%	0.225	v = 15.0%
	12587		0.304	
	10437		0.247	
	11715		0.223	
	10298		0.198	
	4865		0.323	
	6343		0.284	
	6788		0.278	
	7241	$\mu = 5820$	0.250	$\mu = 0.279$
40	4804	s = 1180	0.296	s = 0.032
	4842	v = 20.3%	0.287	v = 11.3%
	6595		0.240	
	4444		0.328	
	7498		0.239	
	4775		0.263	

Table C.4: Resilient Modulus Test Results for Field RMP Samples					
Location	Test Temperature (°C)	Modulus (MPa)	[Statistics]	Poisson's Ratio	[Statistics]
		24389	μ = 21696	0.124	μ = 0.152
Altus	5	18394	s = 3043	0.226	s = 0.064
		22304	v = 14.0%	0.107	v = 42.3%
		13262	μ = 10310	0.300	μ = 0.237
Altus	25	8931	s = 2558	0.273	s = 0.086
		8737	v = 24.8%	0.139	v = 36.4%
		5513	μ = 4975	0.268	μ = 0.243
Altus	40	4739	s = 467	0.171	s = 0.063
		4674	v = 9.4%	0.290	v = 26.1%
		21492	μ = 21356	0.166	μ = 0.196
McChord	5	16217	s = 5073	0.185	s = 0.037
		26360	v = 23.8%	0.237	v = 18.8%
		11050	μ = 8559	0.230	μ = 0.294
McChord	25	6565	s = 2284	0.180	s = 0.157
		8061	v = 26.7%	0.473	v = 53.3%
		4495	μ = 4181	0.328	μ = 0.300
McChord	40	1743	s = 2298	0.103	s = 0.185
		6306	v = 55.0%	0.469	v = 61.5%

References

Roffe, Jean C., 1989a. "Salviacim - Introducing the Pavement," Jean Lefebvre Enterprise, Paris, France.

Jean Lefebvre Enterprise, 1990. "Salviacim - Principal Productions, French and Foreign," Paris, France.

Ahlrich, R. C. and Anderton, G. L., 1991a. "Evaluation of Resin Modified Paving Process," Transportation Research Record 1317, National Research Council, Washington, DC.

Rone, Carlton L., 1976. "Evaluation of Salviacim," Miscellaneous Paper S-76-20, U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, MS.

Ahlrich, R. C. and Anderton, G. L., 1991b. "Construction and Evaluation of Resin Modified Pavement," Technical Report GL-91-13, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Ahlrich, R. C. and Anderton, G. L., 1993. "Resin Modified Pavement in Airfield Applications," Proceedings of American Society of Civil Engineers Specialty Conference on Airport Pavement Innovations - Theory to Practice, ASCE, New York, NY.

Roffe, Jean C., 1989b. "Salviacim (Annex 1) - The Paving Process," Jean Lefebvre Enterprise, Paris, France.

Headquarters, Department of the Army, 1997a. "Use of Resin Modified Pavement (RMP)," Engineer Technical Letter ETL 1110-1-177, Washington, DC.

Headquarters, Department of the Army, 1997b. "Resin Modified Pavement Surfacing Material," Corps of Engineers Guide Specification CEGS-02746, Washington, DC.

Anderton. Gary L., 1996. "User's Guide: Resin Modified Pavement," Miscellaneous Paper GL-96-7, U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, MS.

Blight, G. E., 1984. "The Properties of Salviacim: A Bituminous Bound Cemented Paving Material," TARMAC Roadstone SA (Pty) Limited, Johannesburg, South Africa.

TARMAC Quarry Products, 1986. "Salviacim - Technical Evaluation for Tarmac Roadstone Holdings Limited," Elstree Herts, England.

Al-Qadi, I. L., Prowell, B. D., Weyers, R. E., Dutter, T., Goura, H., and Burke, N., 1993. "Concrete Bridge Protection and Rehabilitation: Chemical and Physical Techniques, Corrosion Inhibitors and Polymers," Report No. SHRP-S-666, Strategic Highway Research Program, National Research Council, Washington, DC.

Packard, Robert, 1996. "The Growth of Whitetopping," Concrete Pavement Progress, Vol. 36, No.1, American Concrete Pavement Association, Arlington Heights, IL.

Speakman, J. and Scott, H. N., 1996. "Ultra-Thin, Fiber-Reinforced Concrete Overlays for Urban Intersections," Transportation Research Record 1532, National Research Council, Washington DC.

Mack, Jim, 1996. "Ultra-Thin Whitetopping the World," Concrete Pavement Progress, Vol. 36, No.2, American Concrete Pavement Association, Arlington Heights, IL.

Mowris, Susan, 1995. "Whitetopping Restores Air Traffic at Spirit of St. Louis," Concrete Construction, The Aberdeen Group, Addison, IL.

American Society for Testing and Materials, 1996a. "1996 Annual Book of ASTM Standards - Concrete and Aggregates," Vol. 04.02 , Philadelphia, PA.

American Society for Testing and Materials, 1996b. "1996 Annual Book of ASTM Standards - Road and Paving Materials; Vehicle-Pavement Systems," Vol. 04.03, Philadelphia, PA.

Asphalt Institute, 1989. "The Asphalt Handbook," Manual Series No. 4, College Park, MD.

Anderton, Gary L., 1990. "Evaluation of Asphalt Binder Modifiers," Miscellaneous Paper GL-90-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Ahlrich, Randy C., 1996. "Influence of Aggregate Gradation and Particle Shape/Texture on Permanent Deformation of Hot Mix Asphalt Pavements," Technical Report GL-96-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

American Concrete Institute, 1976. "Recommended Practice for Evaluation of Strength Test Results of Concrete," ACI 214-77, Detroit, MI.

Departments of the Army and the Air Force, 1987. "Standard Practice for Concrete Pavements," Army TM 5-822-7, Air Force AFM 88-6, Chap.8, Washington, DC.

Melis, L. M., Meyer, A. H., and Fowler, D. W., 1985. "An Evaluation of Tensile Strength Testing," Research Report 432-1F, Center for Transportation Research, University of Texas at Austin, Austin, TX.

Carrasquillo, Peggy M., 1994. "Concrete Strength Testing," STP 169C Concrete and Concrete Making Materials, American Society for Testing and Materials, Philadelphia, PA.

Mindess, S. and Young, F. J., 1981. "Concrete," p381, Prentiss-Hall, Inc., Englewood Cliffs, NJ.

Yoder, E. J. and Witczak, M. W., 1975. "Principals of Pavement Design," Second Edition, John Wiley and Sons, Inc., New York, NY.

American Association of State Highway and Transportation Officials (AASHTO), 1996. AASHTO Provisional Standards, Washington, DC.

Roque, R. and Buttlar, W. G., 1995. "The Development of a Measurement and Analysis System to Accurately Determine Asphalt Concrete Properties Using the Indirect Tensile Mode," Journal of the Association of Asphalt Paving Technologies, Vol. 61, pp 304-332, Bookcrafters, Inc., Chelsea, MI.

Roberts, F. L., Kandhal, P. S., Brown, E. R., Lee, D. Y., and Kennedy, T. W., 1991. "Hot Mix Asphalt Materials, Mixture Design and Construction," p 228, NAPA Education Foundation, Lanham, MD.

Janoo, V., Pellinen, T., Christensen, D., and Von Quintus, H., 1995. "Evaluation of the Low-Temperature Cracking Model in SUPERPAVE," Contract Report DTFH61-95-C-00100, Task D.1.b, Work Element No. 15, Strategic Highway Research Program, National Research Council, Washington, DC.

U.S. Army Corps of Engineers, 1993. "Test Method for Coefficient of Linear Thermal Expansion," CRD-C 39-81, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Al-Qadi, I. L., Prowell, B. D., Weyers, R. E., Duttas, T., and Gouru, H., 1993. "Concrete Bridge Protection and Rehabilitation: Chemical and Physical Techniques," SHRP-S-666, Strategic Highway Research Program, National Research Council, Washington, DC.

Al-Qadi, I. L., Gouru, H., and Weyers, R. E., 1994. "Asphalt Portland Cement Concrete Composite: Laboratory Evaluation," American Society of Civil Engineers Journal of Transportation Engineering, Vol. 120, New York, NY.

Bilodeau, A. and Malhotra, V. M., 1997. "Deicing Salt Scaling Resistance of Concrete Incorporating Supplementary Cementing Materials :CANMET Research," Proceedings of the International Workshop on the Resistance of Concrete to Scaling Due to Freezing in the Presence of De-Icing Salts, Sainte-Foy, Quebec, Canada.

Litvan, G.G., MacInnis, C., and Grattan-Bellew, P. E., 1980. "Cooperative Test Program for Precast Concrete Paving Elements in Durability of Building Materials and Components," ASTM Special Technical Publication STP-691, P. J. Sereda and G. G. Litvan Ed., pp. 560-573, American Society for Testing and Materials, Philadelphia, PA.

Monismith, Carl. L., 1994. "Fatigue Response of Asphalt-Aggregate Mixes," SHRP-A-404, Strategic Highway Research Program, National Research Council, Washington, DC.

Federal Aviation Administration, 1991. "Measurement, Construction, and Maintenance of Skid Resistant Airport Pavement Surfaces," FAA AC 150/5320-12B, Washington, DC.

Chou, Y. T., 1977. "Engineering Behavior of Pavement Materials: State of the Art," Technical Report S-77-9, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Departments of the Army and the Air Force, 1989. "Flexible Pavement Design for Airfields (Elastic Layered Method)," Technical Manual TM 5-825-2-1/AFM 88-6, Chap. 2, Section A, Washington, DC.

Mathews, J. W., Monismith, C. L., and Craus, J., 1993. "Investigation of Laboratory Fatigue Testing Procedures for Asphalt Aggregate Mixtures," American Society of Civil Engineers Journal of Transportation Engineering, Vol. 119, No. 4, New York, NY.

Tayebali, A. A., Deacon, J. A., and Monismith, C. L., 1996. "Development and Evaluation of a Dynamic Flexural Beam Fatigue Test System," Paper No. 960587, Transportation Research Board, Washington, DC.

Vita

Gary Lee Anderton was born in Vicksburg, Mississippi, on October 4, 1961, the son of Louise Hadad Anderton and Elmer Devello Anderton. After earning his diploma from Vicksburg High School in 1979, he entered Mississippi State University where he earned a Bachelor of Science degree in Petroleum Engineering in 1983. He then worked for three years in the petroleum industry for Camco, International out of offices in New Orleans and Lafayette, Louisiana. In 1986, he returned to Vicksburg to begin his current career as a research civil engineer at the U.S. Army Engineer Waterways Experiment Station (WES), specializing in pavement materials and construction technologies. While working at WES, he earned a Master of Science degree in civil engineering from Mississippi State University in 1991. In 1994, he entered the Graduate School of the University of Texas as part of a long-term training assignment. He has authored many technical reports, conference papers, and articles in the pavements field, with over forty publications to his credit.

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14. ABSTRACT

The research study described in this report focuses on determining the engineering properties of the resin modified pavement (RMP) material relating to pavement performance and then developing a rational mechanistic design procedure to replace the current empirical design procedure. A detailed description of RMP is provided, including a review of the available literature on this relatively new pavement technology. Field evaluations of four existing and two new RMP project sites were made to assess critical failure modes and to obtain pavement samples for subsequent laboratory testing. Various engineering properties of laboratory-produced and field-recovered samples of RMP were measured and analyzed. The engineering properties evaluated included those relating to the material's stiffness, strength, thermal properties, and traffic-related properties. Comparisons of these data to typical values for asphalt concrete and Portland cement concrete were made to relate the physical nature of RMP to more common pavement surfacing materials. A mechanistic design procedure was developed to determine appropriate thickness profiles of RMP, using stiffness and fatigue properties determined by this study. The design procedure is based on the U.S. Army Corps of Engineers layered elastic method for airfield flexible pavements. The WESPAVE computer program was used to demonstrate the new design procedure for a hypothetical airfield apron design.

15. SUBJECT TERMS

Asphalt concrete, Cement grout, Composite pavement, Flexible pavement, Pavement design, Portland cement concrete, Resin modified pavement, Salviacim

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