COMPENDIUM OF RESEARCH ACTIVITIES TO DATE: EVALUATION OF THE PERFORMANCE OF TEXAS ASPHALT CONCRETE PAVEMENTS MADE WITH DIFFERENT COARSE AGGREGATES

B. F. McCullough
D. G. Zollinger

Research Compendium 1244-1 (Vol. 1)

Prepared by the

CENTER FOR TRANSPORTATION RESEARCH
Bureau of Engineering Research
THE UNIVERSITY OF TEXAS AT AUSTIN

and the

TEXAS TRANSPORTATION INSTITUTE
TEXAS A&M UNIVERSITY

May 1993

COMPENDIUM OF RESEARCH ACTIVITIES TO DATE: EVALUATION OF THE PERFORMANCE OF TEXAS ASPHALT CONCRETE PAVEMENTS MADE WITH DIFFERENT COARSE AGGREGATES

by

B. F. McCullough
D. G. Zollinger

Research Compendium 1244-1 (Vol. 1)

Evaluation of the Performance of Texas Pavements Made with Different Coarse Aggregates
Research Project 2/3-12D-90/4-1244

conducted for the

Texas Department of Transportation

in cooperation with the
U.S. Department of Transportation
Federal Highway Administration

by the

CENTER FOR TRANSPORTATION RESEARCH
Bureau of Engineering Research
THE UNIVERSITY OF TEXAS AT AUSTIN
and the
TEXAS TRANSPORTATION INSTITUTE
TEXAS A&M UNIVERSITY

May 1993

Study Supervisor:
Dr. B. F. McCullough, P.E. (Texas No. 19914)

PREFACE

This is the Compendium of Information (Volume 1) "Evaluation of the Performance of Texas Asphalt Concrete Pavements made with Different Coarse Aggregates." This compendium is summarized by the fifth report for Research Project 1244, "Preliminary Research Findings on the Affect of Coarse Aggregate on the Performance of Portland Cement Concrete Paving." This compendium of information was the result of a joint effort between the University of Texas at Austin, Center for Transportation Research and the Texas Transportation Institute.

In this compendium, the various areas of research that have been performed to date on aggregates used for both asphalt and Portland Cement Concrete pavements are documented. Field and laboratory investigations have shown that significantly different pavement performance con be expected for aggregates with relatively high thermal coefficients. With regard to concrete pavements, environmental conditions at the time of placement have also been closely tied to performance.

We would like to thank the staff of the Texas Department of Transportation for their support throughout this study. Their interest and enthusiasm in this project has resulted in numerous pavement test sections being constructed in Houston, LaPorte, Cypress, and Texarkana, Texas. These well documented test sections will provide Department engineers with excellent data on the long term performance of pavements constructed with different aggregates.

ABSTRACT

For many years, engineers have recognized the different performance characteristics of asphalt and concrete pavements constructed with different coarse and fine aggregates. Past research has shown the importance of monitoring these aggregates for physical and chemical properties such as abrasion resistance, polish value, gradation, soundness, fineness modulus, specific gravity, and absorption. Using various tests to determine these aggregate properties, engineers are able to screen aggregates before they are ever used for pavement construction. Unfortunately, this selection of tests does not insure the long term performance of pavements in the field as illustrated by the numerous pavement failures at early age. This research attempts to address these shortfalls in the quality control and pavement design processes related to aggregates used in construction. This research has been comprehensive in the area of pavement aggregate. Numerous Master's Thesis' and Ph.D. dissertations have been completed under the auspices of this study. These include work performed both at Texas A & M University and the University of Texas at Austin. As a result, the principal investigators responsible for this study, felt it was necessary to pull together all of the work that has been performed to date in the form of a compendium. The compendium is comprehensive and includes detailed research activities that have been conducted to date.

SUMMARY

This compendium of information presents all of the research activities that have been completed to date for this study. This compendium of information is meant to illustrate the many of the research activities that have been completed to date both at Texas A & M University and the University of Texas at Austin. The compendium of information focuses on research activities related to aggregates used in the construction of asphalt concrete pavements.

This compendium of information also addresses research activities related to aggregates used in the construction of Portland cement concrete pavements. These include specific chapters addressing topics such as field investigations of spalling and punch-out distresses in continuously reinforced and jointed pavements, aggregate shape characterization using fractals, and determination of saw-cut depth using fracture analysis. Some of these activities relate directly to improving pavement performance regardless of aggregate type used for construction.

Work to date has also focused on identifying significant factors that affect the performance of asphalt and concrete pavements. Significant accomplishments have been made in this area and early recommendations have been made to the Texas Department of Transportation regarding implementation of these findings. The details of these recommendations are documented in another 1244 report regarding field implementation of significant findings, to be published at a later date.

IMPLEMENTATION STATEMENT

The findings discussed within this report will help optimize designs of Portland cement concrete pavements. Improvements in coarse aggregate selection can, by offsetting various distress manifestations, lead to improved pavement performance. The field data collected and evaluated in this report can also potentially serve as the basis for improving existing design equations. These equations take into consideration the studies on determination of saw-cut depth using fractal analysis. Finally, improvement in material selection for pavement construction can translate into a direct cost benefit to the Department.

TABLE OF CONTENTS - VOLUME I

CHAPTER 1

INTRODUCTION AND BACKGROUND

The Texas Department of Transportation (TxDOT) has a total centerline mileage of approximately 73,000 miles of pavements. Future design plans call for the construction of many road miles of new pavements. In the past, the design and construction of asphaltic concrete (AC) and portland cement concrete (PCC) pavements in Texas did not take into account the variation in pavement material properties that may be attributed to the use of different coarse aggregate types.

1.1 ORIGINATION OF PROBLEM STATEMENT

In the case of PCC pavements, principally continuously reinforced concrete (CRC) pavements, the design and construction is based on the premise that the concrete volume changes are accounted for by a random occurrence of transverse cracks that are allowed to develop as a result of shrinkage and temperature changes. The movement at the cracks is minimized by longitudinal steel that is placed in the slab to insure a narrow crack width. Thus, the crack pattern involving the crack spacing and the crack width, is one of the most important physical aspects of the CRC pavement design. Consequently, the design crack spacing becomes one of the focal points of the design process and inherently is the key to achieving equality in the performance of the pavement with respect to the type of aggregate.

As for asphalt pavements one of the primary concerns is the development of rutting. Rutting in asphalt pavements is caused by the progressive movement of materials under repeated loads, either in the pavement layers and/or in the underlaying base or subgrade. Several material factors have been noted to affect the development of rutting such as the degree of consolidation, amount and type of asphalt, and aggregate gradation and type.

Unfortunately, the substantial effect of the coarse aggregate on pavement performance has not been fully recognized in the design-construction sequence. Several physical properties of paving materials may vary with coarse aggregate type such as the modulus of elasticity, the coefficient of contraction and expansion, aggregate bonding characteristics, and the tensile strength. All of these, in turn, can influence pavement performance and efforts have been made in the past to account for these influences in design. For instance, in 1981 as a result of the findings recommended in Report 177-22F, "Summary and Recommendations for the Implementation of Rigid Pavement Design, Construction, and Rehabilitation Techniques," a new design procedure was issued by the Texas DOT Highway Design Division that permits a more rational analysis of all the factors influencing CRC pavement performance.

Although the design process for CRC pavement now recognizes the performance difference of the coarse aggregate types, the selection of coarse-aggregate types used during construction is left to the contractor by the present specifications. Hence, as long as the aggregate meets the gradation and physical requirements, the basic assumption is that all aggregates are equivalent in performance and thus, are acceptable. However, field performance has demonstrated that the pavements constructed with different coarse aggregate types exhibit a substantial difference in crack pattern and performance life, even though it is assumed that they will have the same life.

An increasing variety of coarse aggregate types are becoming available for paving in Texas. However, most of the pavements are constructed with materials using either a (1) crushed limestone aggregate or (2) siliceous river gravel. During the competitive bidding process, a contractor generally selects the aggregate type, based on competitive prices received from the various aggregate suppliers. The contractor will then construct the pavement required in the project plan with the coarse aggregate of his own choice even though field performance indicates this is not the ideal approach.

A primary focus of this study is to investigate the potential to reduce the effect of the coarse aggregate type on the performance of CRC pavement in Texas with respect to the development of the cracking pattern and examine how this may be compensated for in other phases of the thickness design process. Consideration of the effect of crack width (along with load transfer), steel design, and other factors affecting the performance of CRC pavement and the development of punchout distress should lead to design modifications which should equalize these performance factors in terms of coarse aggregate type.

The concrete pavement portion of this research project is related to the understanding of crack control which addresses the development of cracking in CRC pavement at a given interval from different perspectives. CRC pavement performance should be improved through implementation of an improved methodology to control the cracking pattern that develops in the pavement. This project accomplishes this task by examining the material characteristics and describing it in terms of different coarse aggregates.

There are presently no standard tests or materials specifications that directly address aggregate properties such as type and, particle shape It is well known, however, that these properties directly affect asphalt concrete pavement performance. Mixture tests such as Hveem stability or axial compression provide an indirect measure of these aggregate qualities. The relatively high incidence of pavement failure, which many times appears related to deficient aggregate quality, indicates that there is a need to develop test methods and establish acceptance criteria that can be used to rate aggregates.

The significance of this work is to address the issue of achieving equal pavement performance using different coarse aggregates as construction materials in Texas. The focus of this study is to organize field data and conduct analyses based on techniques that will permit the inference of conclusions on the type, extent and practical difference of performance of various aggregates used in Texas for pavement construction. These results should provide the basis for decisions on establishing policies that reflect the ultimate goal of providing pavements to the public that will provide equal performance without respect to the material used for design. Due to the long range implications and scope of this study both the Center for Transportation Research (CTR) at the University of Texas and the Texas Transportation Institute (TTI) at Texas A&M University are participating in this project.

1.2 STUDY OBJECTIVES AND OVERVIEW OF PROJECT WORK PLAN

The progress to date on this study has been focused on the study of the effect of the coarse aggregate on the performance of CRC, jointed concrete pavement, and hot-mix asphalt concrete (HMAC) pavement. Various distresses in these types of pavements have been selected to for specific research to investigate associative causes in an effort to determine corrective measures. The nature of these measures are characteristic of the coarse aggregate used in the pavement and whether the material is concrete or asphalt. The objectives of this study can be described as follows:

1. To collect and analyze information available from field and laboratory evaluations that may lead to the description and explanation of the effects that coarse aggregate type may have on the performance of AC and PCC pavements in Texas.
2. To identify and focus on significant pavement distresses and determine how the failure modes and the distress manifestations are related to the types and physical characteristics of aggregates used in Texas.
3. To develop model improvements to account for aggregate-related distresses not presently accounted for.
4. To propose alternative design and/or construction methods (with industry input) for reducing identified pavement distress and improving pavement performance using practical solution approaches that can be implemented by Texas DOT.

The project is organized into three phases of work consisting of a field phase, a laboratory phase, and a design model improvement phase. The field and laboratory phases of this study are designed to determine relationships between coarse aggregate characteristics and specific modes of pavement distress related to pavement cracking, spalling, and rutting. The field investigation includes or will include existing asphalt and concrete pavements comprised of different aggregates types. Laboratory testing of pavement materials have aided in determining the nature of pavement distress. The laboratory program has undertaken experiments to verify findings from the field study which can be used to develop modifications to the design guidelines in terms of aggregate characteristics.

1.2.1 Field Study Phase

This phase of the project emphasizes two areas: (1) the investigation of specific pavement distress as a function of the coarse aggregate type, and (2) the monitoring of experimental pavement sections. The first area of emphasis considers as a minimum the coarse aggregate factors affecting the development of pavement spalling in CRC (and jointed if applicable) pavement and pavement rutting in asphalt concrete pavement. However, it is noted that the influence of the coarse aggregate in asphalt concrete pavement cannot be completely isolated from the often overriding effects of the fine aggregate and filler size particles. The tasks and subtasks which are associated with this phase of work can be found in the original project proposal.

1.2.2 Laboratory Phase

This phase of the project consists of laboratory work for both concrete and asphalt pavements. The concrete laboratory work provides verification data for relationships between concrete mechanical characteristics and environmental conditions and the time of construction.

There are presently no standard tests nor materials specifications that directly address asphalt aggregate properties such as particle shape and surface texture in asphalt pavements. It is well known, however, that these properties directly affect asphalt concrete pavement performance. Mixture tests such as Hveem Stability or axial compression provide an indirect measure of these aggregate qualities. Nevertheless, there is a need to develop acceptance criteria that can be used to eliminate substandard aggregates.

1.2.3 Design Model Improvement Phase

When new materials or test sections are placed, performance information is needed in the shortest possible time. Often, only one or two years are available before decisions must be made based on the performance.

A model used in this project is available which predicts the performance of CRC Pavement. The approach requires the input of relatively precise data, but predictions of the extent of transverse cracking, failures and repairs result. The model was developed using actual "CRC pavement condition data, but the availability of a large amount of new CRC pavement condition data will permit the calibration and an improved predictor of CRC pavement performance. The work in this task will perform the calibration and model improvement.

Other tasks in the design model improvement phase include development of a pavement spalling model, improvements to the pavement cracking model, and asphaltic concrete pavement model improvements. The results will also be verified through detailed economic analysis and field implementation.

CHAPTER 2

SYNOPSIS OF PAST RESEARCH AND PRELIMINARY FINDINGS

Past research efforts on pavement performance has resulted in the development of data bases which provide some insight into pavement behavior characteristics. Factors included in each data base are closely examined and evaluated with respect to specific pavement distresses considered in this study.

2.1 RUTTING IN HMAC PAVEMENTS

Rutting in asphalt pavement develops due to accumulated movement of materials under repeated loads, either in the pavement layers and/or in the underlaying base or subgrade. Several factors have been noted to affect the development of rutting and one factor is tire pressure. As the tire pressure increases the potential for rutting also increases. Tire pressures of 75 to 80 psi are generally used for pavement design but surveys indicate that in Texas averaging 110 psi with values ranging as high as 155 psi (Ref 2.1). The future performance of asphalt mixes which provided satisfactory results under tire pressures of the past is unpredictable with respect to the noted recent increase in truck tire pressures. It should also be pointed out that the future performance of new mix designs (to account for the increased tire pressures) will be equally unpredictable unless adequate testing and theoretical models are developed to characterize the significant controlling factors and associated variability.

Another factor affecting the occurrence of rutting in asphalt pavements is the degree of consolidation. Further compaction, after construction of the pavement, in the asphalt layer due to traffic will lead to a depression in the wheel paths. This additional compaction can be caused by inadequate compaction during construction, under estimation of the design traffic, and/or poor asphalt concrete design. Poor compaction may lead to rutting because of the kneading action of traffic loads concentrated in the wheel paths. This action can also cause shoving and heaving in the pavement surface adjacent to the wheel path.

A phenomenon related to shoving is plastic flow. One of the most common problems associated with rutting is the use of excess asphalt in the mix often added to facilitate achieving the required compaction (density). Asphalt cement may flow under repeated load stress leading to permanent deformation. Insufficient asphalt will lead to cracking or raveling.

Due to the nature of plastic flow in asphalt concrete, permanent deformation may be very small for one load application but after many applications, will develop into considerable ruts. It should also be noted that load stresses applied over a long period of time at high temperatures can cause greater permanent deformation. That is, slow-moving or stationary traffic will produce more rutting than fast-moving traffic.

Plastic flow is also affected by the size, shape, and texture of the aggregate in a bituminous mixture. Small sizes, rounded shapes, and smooth textures tend to increase rutting. Ideally, the aggregate "skeleton" should exhibit stone-on-stone contact such that the traffic loads will be directly supported by the aggregate and minimize the load carried by the asphalt cement. High concentrations of natural (uncrushed) sand can be a large contributor to rut development in asphalt concrete pavements. The Federal Highway Administration (FHWA) recommends that natural sand content be limited to 15 to 20 percent of the total weight of the mix tot minimize the rutting potential on high-volume roadways (Ref 2.2). However, adequate test methods have not been developed to objectively evaluate the angularity or the surface texture of aggregates.

The binder, which consists of the asphalt and the fine aggregate (minus no. 200 sieve size) also plays a role in plastic flow of the asphalt mix. If the quantity of the fines are too small, the mixture may lack stability and rutting potential will increase. If the quantity of fines are too high (in conjunction with additional asphalt cement required to coat these particles) leading to over filling of the voids in the mix, rutting potential is again increased. Harder asphalt will provide more rut resistant pavements, however, the effect of harder asphalt will be less significant with respect to rut resistance if appropriately graded, high quality aggregates with sharp edges and high surface friction are used in the mix.

Design of dense graded asphalt concrete mixes must be balanced between rutting and cracking and, historically, this balancing has not been an easy task. Asphalt pavements which develop a minimum of rutting or cracking can be designed and built but cost factors must also be considered since mixes similar to this tend to be economically prohibitive. In order to satisfy the economic requirements or, that is, pave as much roadway as possible with existing funds, local materials (often of marginal quality) must be used whenever possible. This common practice will occasionally result in asphalt mixtures of marginal quality. Pavement design engineers are required to optimize all these (and other) factors to provide a reasonable balance between rutting, cracking and economy. It is unreasonable to assume that rutting to some degree is not going to develop over the design life of the pavement since several factors influence rutting. It should be pointed out, however, that increasing asphalt pavement layer thickness would not improve resistance to rutting in the asphalt layer. However, some positive considerations are listed below as:

- Minimum voids in the mineral aggregate (VMA) requirement,
- Increased design air void content of the mixture,
- Replaced a portion of the natural (uncrushed) sand with manufactured (crushed) sand,
- Required the sue of AC-20, thus eliminating AC-10, and
- Eliminated the use of recycled asphalt pavement.

All of these specification changes would be accepted by asphalt technologists as positive moves to address rutting. In addition, these changes will result in significant increases in the cost of materials as well as possible increases in pavement construction, since these mixtures will be more difficult to compact.

2.2 ASPHALT PAVEMENT PERFORMANCE

A portion of this study was dedicated to the investigation of available databases documenting asphalt pavement performance. From these databases, selected pavement section was chosen to identify additional data such as layer thicknesses and time of layer construction. The selected sections can then be considered for specific rutting study.

Of the available data bases, the TTI flexible pavement database and the Texas DOT PES database were considered as the candidate databases for detailed review. The PES database, though comprehensive in terms of number of test sections on which pavement condition information was available, had little or no information on the pavement construction record, which was crucial in finding additional information such as type and source of aggregates used. Therefore, it was determined to include the TTI flexible pavement database.

The TTI flexible pavement database contains details on the construction, maintenance and rehabilitation records of test sections and also the types and sources of aggregates used. However, the information on the aggregates is not extensive, and assistance had to be sought from districts of the Texas DOT in order complete those records.

A detailed summary of information important to this research project which are available in the TTI flexible pavement database and the PES database are listed in Appendix A.

One of the major difficulties encountered in doing this part of the study was the lack of information on test sections in one database. Most of the pavement sections were constructed several years ago where the records of their construction details were often not available even in the respective districts. This meant that such information could only be obtained from folders stored in a TxDOT warehouse in Austin. Fortunately, the TTI flexible pavement database had information collected on test sections which proved to be most useful in trying to locate information from districts. Most of the construction information could not be traced from information available from the PES database because they were stored under project numbers which were no longer available.

A salient feature of the history of most test sections which have been in service for a long time, is that repeated maintenance, rehabilitation, and reconstruction have been performed on them. In many instances, it was very difficult to single out the effect of a particular surface, and consequently, a single aggregate type or source. Consequently, the test sections need to be fairly recent, and need to be analyzed at least prior to any major maintenance, rehabilitation, or reconstruction performed on them. This would enable one to isolate the various factors involved much more effectively.

The following table indicates the summary of action regarding the correspondence made with districts requesting further information on test sections.

The test sections for this study were selected from available test sections in the TTI flexible pavement database based on the thickness of the asphalt concrete pavement surface. The sections were selected such that the thickness was between and 2 and 4.5 inches to ensure that they would have the potential of at least the two major modes of failure, namely rutting and cracking. The selected test sections were spread across the state in 18 of the 24 districts.

2.3 REFERENCES

1. Roberts, F. L., Tielking, J. T., Middleton, D., Lytton, R. L., and Tseng, K., "Effects of Tire Pressures on Flexible Pavements," Report 372-1F, Texas Transportation Institute, Texas A&M University, December 1985.

2. Heing, R. E., "Asphalt Concrete Mix Design and Field Control," Technical Advisory T5040.27, Federal Highway Administration, Department of Transportation, Washington, D. C., March 10, 1988.

APPENDIX A

A.1 TTI FLEXIBLE PAVEMENT DATA BASE

The information available from the TTI flexible pavement database is classified into the following information groups.

Location Information

Environmental Information

Traffic Information

Structural Section Information

Pavement Condition Information

Skid Number Measurements

Serviceability Index measurements

Dynaflect Deflection Measurements

Of these groups, the most useful information are contained in the structural section information and pavement condition survey information. The information included under these two categories are summarized below.

A.1.1 STRUCTURAL SECTION INFORMATION

Layer No.

Structure No.

Description of Layer

- Layer Classification (e.g. Surface or Base or Subgrade)
- Layer Type (e.g. Clay, PCC, HMAC, SC etc.)

Work Done to Pavement

- Type of Work (e.g. New Construction,Widening,Maintenance etc.)
- Date (Mo./Year)
- Aggregate/Soil Type
- Item #, Classification, Type

Aggregate Rate

Admixtures (or Asphalt Content)

Applied Thickness and Rate of Application

Texas Tri-axial Class, Liquid Limit, and Plasticity Index (for applicable soils)

A.1.2 PAVEMENT CONDITION SURVEY

A.1.2.1 Pavement Rating

Pavement Rating Score (PRS)

Rutting (measured in percent of area with rutting)

Level 1	-	1 to 15 percent
Level 2	-	16 to 30 percent
Level 3	-	greater than 30 percent

Three levels of severity : Slight, Moderate and Severe

Flushing (measured in percent of area with flushing)

Level 1	-	1 to 15 percent
Level 2	-	16 to 30 percent
Level 3	-	greater than 30 percent

Three levels of severity : Slight, Moderate and Severe

Corrugation (measured in percent of area with corrugations)

Level 1	-	1 to 15 percent
Level 2	-	16 to 30 percent
Level 3	-	greater than 30 percent

Three levels of severity : Slight, Moderate and Severe

Ravelling (measured in percent of area with corrugations)

Level 1	-	1 to 15 percent
Level 2	-	16 to 30 percent
Level 3	-	greater than 30 percent

Three levels of severity : Slight, Moderate and Severe

Alligator Cracking (measured in percent of area with alligator cracking)

Level 1	-	1 to 5 percent
Level 2	-	6 to 25 percent
Level 3	-	greater than 25 percent

Three levels of severity : Slight, Moderate and Severe

Longitudinal Cracking (measured in lineal ft. per station per lane)

Level 1	-	10 to 99 lineal ft.
Level 2	-	100 to 199 lineal ft.
Level 3	-	greater than 200 lineal ft.

Three levels of severity : Slight, Moderate and Severe

Transverse Cracking (measured in number of cracks per station)

Level 1	-	1 to 4 cracks
Level 2	-	5 to 9 cracks
Level 3	-	greater than 10 cracks

Three levels of severity : Slight, Moderate and Severe

Patching (measured as a percent of total area)

Level 1	-	1 to 5 percent
Level 2	-	6 to 15 percent
Level 3	-	greater than 15 cracks

Three levels of severity : Good, Fair and Poor

Failures per Mile (measured in number of failures per mile)

Level 1	-	1 to 5 percent
Level 2	-	6 to 10 percent
Level 3	-	greater than 10 cracks

State of Cracks (Sealed, Partially Sealed and Not Sealed)

A.1.2.2 Rating Scores

This database also has rating scores for the following categories.

Shoulder Rating for paved shoulders

Shoulder Rating for unpaved shoulders

Roadside Rating

Drainage Rating

Traffic Service Rating

Pavement Urgency (Scale of 1 to 10 with 10 being most urgent)

A.2 TEXAS PES DATABASE

This data base provided general information regarding the administration details and location information of test sections, and also pavement condition information. These information have been compiled separately for asphalt concrete, continuously reinforce concrete, jointed reinforced concrete pavements.

Information available in the PES database for flexible pavements are classified in to the categories of general information, and pavement condition information.

Since the pavement condition information is the most important aspect of the survey, its salient elements are indicated below.

Rutting

* 3 severity levels.

0 for	<	1/2" of rutting
1 for		1/2" to 1"
2 for	>	1"

* Rutting is measured as a percent of wheel paths.
  (Both area and severity are recorded)

* Guidelines :

1 -	25 %	One wheel path(WP) discontinuous
25 -	50 %	One WP continuous and none in other WP's
>	50 %	One WP continuous and some in other WP's, or, Two WP's discontinuous but adding up to more than one continuous WP.

Patching

* Measured as a percentage of the total lane surface area (for the lane with most distress)
* Guidelines :

1 -	10 %	Sporadic patching, but < 10 % of section area.
11 -	50 %	Large areas of patching at regular intervals.
>	50 %	Large areas of patching, some new patches joining old patches, patching almost always.

* All patches are recorded except full roadway treatments greater than 500 ft.

Failures

* Definitions of failure :
  1. A section of pavement usually less than 20 ft. with surface eroded or badly cracked or depressed.
  2. The areas may contain loose pieces of material and create a hazardous driving condition.
  3. Individual potholes are not taken as failures, but, each 10 potholes per lane mile is a failure.
  4. Failed areas excess of 40 L.ft. are taken as multiple failures for each 40 ft. or fraction thereof.
* Recorded as No. of failures per lane mile (worst lane). Only un-repaired failures are measured.
* Failures adequately repaired are rated as a patch.
* Categories :

1 -	5 failures
6 -	10 failures
>	10 failures

Block cracking

* Interconnecting cracks that divide pavement into pieces approx. 1'x1' to 10'x10' in size.
* Measured as a percentage of total lane surface area.
* Only cracks wider than 1/8 " is considered.
* Guidelines :

1 -	10 %	Only localized areas of block cracking.
11 -	50 %	Regular areas of block cracking present.
>	50 %	Extensive areas of block cracking.

Alligator cracking

* Measured as percent of total wheel paths.
* Guidelines :

1 -	10 %	Only localized areas of cracking. Total length of wheel path showing this distress is less than 1000 ft.
11 -	50 %	Regular areas of alligator cracking found, but total length is less than half of total wheel path length.
>	50 %	Extensive alligator cracking found. Total length greater than one wheel path.

Longitudinal cracks

* Definition :

  These are cracks or breaks approx. parallel to the pavement centerline. (e.g. edge cracks, edge joint cracks between the pavement and the shoulder, lane joint cracks, reflection cracks, cracks created by volume changes in the subgrade.
* Recorded when there is;

  1 evidence of spalling or pumping,

  2 crack width is greater than 1/8", or

  3 the crack is sealed.
* Number of lineal feet of cracking per station (100 ft.) is measured.
* Categories :

10 -	99 ft.	average lineal feet of crack.
100 -	200 ft.	
>	200 ft.	

Transverse cracking

* These are to be recorded when there is

  1 evidence of spalling or pumping,

  2 crack width is greater than 1/8", or

  3 the crack is sealed.
* No. of full lane width cracks are counted. Cracks that do not extend full lane width are summed until they equal the lane width.
* Categories :

1 -	4	average no. of cracks per station.
5 -	10	average no. of cracks per station.
>	10	average no. of cracks per station.

No information is available on aggregates used.

APPENDIX B

DETAILS OF TEST SECTIONS FOR WHICH INFORMATION WERE REQUESTED FROM DISTRICTS

SID-Highway	39 IH 30			
Cont. Sect.	9 13			
District	1			
County	117 Hunt			
Year Month Layer Aggregate and Source	Layer Materials	Center		Job Coarse
Description	Classification	Thickness No.		
1967 8	HMAC Layer	HMAC	2.50	
1967 8	Original Surface	HMAC	1.50	
1984 8	Overlay	HMAC	2.00	
1986 8	Overlay	HMAC	0.75	

SID-Highway	186 US 377			
Cont.-Sect.	80 7			
District	2			
County	220 Tarrant			
Year Month Layer Aggregate and Source		Layer Material	Center	Job Coarse
	Description	Classification	Thickness No.	
1970 8	Overlay	HMAC	1.45	
1980 8	HMAC Layer	HMAC	2.00	
1980 8	HMAC LAyer	HMAC	1.50	

SID-Highway	199 SH 303 S			
Cont.-Sect.	2208 1			
District	2			
County	220 Tarrant			
Year Month Layer Aggregate and Source		Layer Material	Center	Job Coarse
	Description	Classification	Thickness No.	
1968 12	HMAC Layer	HMAC	1.25	
1972 7	Original Surface	HMAC	1.25	
1976 6	Por. Fr. Course	Open Gr.Fr.Course	1.00	
1981 2	Overlay	HMAC	3.50	

SID-Highway	306 IH 40			
Cont.-Sect.	275 4			
District	4			
County	33 Carson			
Year Month Layer Aggregate and Source		Layer Material	Center	Job Coarse
	Description	Classification	Thickness No.	
1971 4	HMAC Layer	HMAC	3.00	
1971 4	Original Surface	HMAC	1.00	
1980 8	Overlay	HMAC	2.00	

SID-Highway	393 IH 40			
Cont.-Sect.	90 3			
District	4			
County	180 Oldham			
Year Month Layer Aggregate and Source		Layer Material	Center	Job Coarse
	Description	Classification	Thickness No.	
1969 2	Part. Milled Sur.	HMAC	0.15	
1969 2	HMAC Layer	HMAC	1.65	
1969 2	HMAC Layer	HMAC	1.35	
1973 9	HMAC Layer	HMAC	1.50	
1979 10	HMAC Layer	HMAC	2.00	
1985 11	Overlay	HMAC	2.00	

SID-Highway	424 US 54			
Cont.-Sect.	238 2			
District	4			
County	104 Hartley			
Year Month Layer Aggregate and Source		Layer Material	Center	Job Coarse
	Description	Classification	Thickness No.	
1972 8	HMAC Layer	HMAC	3.00	
1972 8	Original Surface	HMAC	1.25	
1985 12	Overlay	HMAC	1.50	

SID-Highway	2646 US 84			
Cont.-Sect.	53 5			
District	5			
County	86 Garza			
Year Month Layer Aggregate and Source		Layer Material	Center	Job Coarse
	Description	Classification	Thickness No.	
1971 1	HMAC Layer	HMAC	3.00	
1971 1	Original Surface	HMAC	1.50	
1980 8	Overlay	HMAC	2.00	

SID-Highway	2659 US 84
Cont.-Sect.	53 5
District	5
County	86 Garza

Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1971	1	HMAC Layer	HMAC	3.00		
1971	1	Original Surface	HMAC	1.50		
1980	8	Overlay	HMAC	2.00		
1980	8	Surface Treatment	1-Course ST	0.30		

SID-Highway	2748 US 84
Cont.-Sect.	52 5
District	5
County	140 Lamb

Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1966	1	Surface Treatment	1-Course ST	0.30		
1966	10	Original Surface	HMAC	1.50		
1987	5	HMAC Layer	HMAC	3.00		
1987	5	Overlay	HMAC	1.50		

SID-Highway	602 IH 20
Cont.-Sect.	4 7
District	6
County	69 Ector

Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1970	11	Surface Trt.	2-Course ST	0.32		
1970	11	Surface Treatment	2-Course ST	0.24		
1973	3	HMAC Layer	HMAC	2.05		
1973	3	HMAC Layer	HMAC	1.25		
1977	11	HMAC Layer	HMAC	1.00		
1987	10	Overlay	HMAC	2.75		

SID-Highway	924 IH 35
Cont.-Sect.	15 6
District	9
County	14 Bell

Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1958	8	HMAC Layer	HMAC	4.00		
1967	4	HMAC Layer	HMAC	1.50		
1981	8	Overlay	HMAC	1.50		
1981	8	Overlay	HMAC	1.25		

SID-Highway	966 SH 6
Cont.-Sect.	258 7
District	9
County	18 Hunt

Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1966	9	HMAC Layer	HMAC	2.00		
1966	9	Original Surface	HMAC	1.10		
1974	4	Overlay	HMAC	2.20		
1982	11	Overlay	HMAC	1.50		

SID-Highway	995 IH 35
Cont.-Sect.	14 7
District	9
County	110 Hunt

Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1957	11	HMAC Layer	HMAC	2.70		
1962	8	HMAC Layer	HMAC	1.50		
1967	8	Part.Milled Sur.	HMAC	0.89		
1967	8	HMAC Layer	HMAC	1.00		
1988	8	HMAC Layer	HMAC	3.00		
1988	8	Overlay	HMAC	1.50		
1974	11	Overlay	HMAC	3.50		
1974	11	Overlay	HMAC	0.65		

SID-Highway	1174 US 59
Cont.-Sect.	175 7
District	11
County	174 Nacogdoches

Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1965	8	HMAC Layer	HMAC	1.25		
1975	9	Overlay	HMAC	1.00		
1975	9	HMAC Layer	HMAC	2.00		
1983	7	Overlay	HMAC	1.75		

SID-Highway	1190 US 96
Cont.-Sect.	64 5
District	11
County	202 Sabine

Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1954	11	HMAC Layer	CMRA	1.25		
1961	9	HMAC Layer	CMRA	1.50		
1966	8	HMAC Layer	HMAC	1.50		
1985	6	Overlay	HMAC	1.50		

SID-Highway	1234 SH 35				
Cont.-Sect.	178 3				
District	12				
County	20 Brazoria				
Year Month	Layer Description	Layer Classification	Material Thickness	Center Job No.	Coarse Aggregate and Source
1959 3	HMAC Layer	HMAC	3.00		
1971 10	Original Surface	HMAC	1.00		
1971 10	HMAC Layer	HMAC	1.00		
1982 2	Overlay	HMAC	2.50		

SID-Highway	1263 SH 6				
Cont.-Sect.	192 4				
District	12				
County	85 Galveston				
Year Month	Layer Description	Layer Classification	Material Thickness	Center Job No.	Coarse Aggregate and Source
1966 3	HMAC Layer	HMAC	4.75		
1969 7	HMAC Layer	HMAC	0.75		
1969 7	Original Surface	HMAC	1.25		
1973 8	Overlay	HMAC	1.00		

SID-Highway	1323 US 290				
Cont.-Sect.	50 5				
District	12				
County	237 Galveston				
Year Month	Layer Description	Layer Classification	Material Thickness	Center Job No.	Coarse Aggregate and Source
1952 7	HMAC Layer	HMAC	1.25		
1963 8	HMAC Layer	HMAC	3.50		
1967 6	Original Surface	HMAC	2.00		
1979 8	Overlay	HMAC	1.10		
1979 8	Overlay	HMAC	2.00		

SID-Highway	1412 US 59				
Cont.-Sect.	89 6				
District	13				
County	241 Wharton				
Year Month	Layer Description	Layer Classification	Material Thickness	Center Job No.	Coarse Aggregate and Source
1948 4	HMAC Layer	HMAC	0.75		
1956 6	HMAC Layer	HMCL	1.00		
1963 9	HMAC Layer	HMAC	1.50		
1973 11	HMAC Layer	HMAC	2.50		
1973 11	Overlay	HMAC	1.00		

SID-Highway	1543 IH 10					
Cont.-Sect.	25 2					
District	15					
County	15 Bexar					
Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1968	8	Original Surface	HMAC	1.00		
1978	8	Overlay	HMAC	1.00		
1978	8	Overlay	HMCL	0.40		
1987	3	Overlay	HMAC	1.00		
1987	3	HMAC Layer	HMAC	1.50		

SID-Highway	1556 US 90					
Cont.-Sect.	24 7					
District	15					
County	15 Bexar					
Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1971	4	HMAC Layer	HMAC	4.00		
1971	4	Original Surface	HMAC	1.50		
1983	5	Surface Treatment	HMCL	0.55		

SID-Highway	1603 IH 45					
Cont.-Sect.	17 8					
District	15					
County	142 Bexar					
Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1969	4	HMAC Layer	HMAC	4.50		
1969	4	Original Surface	HMAC	1.50		
1988	8	HMAC Layer	HMAC	1.50		

SID-Highway	3341 IH 35					
Cont.-Sect.	17 8					
District	15					
County	142 La Salle					
Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1969	4	HMAC Layer	HMAC	4.50		
1969	4	Original Surface	HMAC	1.50		
1988	8	Overlay	HMAC	1.50		

SID-Highway	3354 IH 35					
Cont.-Sect.	17 5					
District	15					
County	163 Medina					
Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1967	6	HMAC Layer	HMAC	5.50		
1967	6	Original Surface	HMAC	1.50		

SID-Highway	3367 IH 35
Cont.-Sect.	17 5
District	15
County	163 Medina

Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1968	5	HMAC Layer	HMAC	5.50		
1968	5	Original Surface	HMAC	1.50		

SID-Highway	3370 IH 35
Cont.-Sect.	17 6
District	15
County	83 Frio

Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1968	5	HMAC Layer	HMAC	5.50		
1968	5	Original Surface	HMAC	1.50		

SID-Highway	1687 IH 37
Cont.-Sect.	74 6
District	16
County	205 San Patricio

Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1966	9	Surface Treatment	1-Course ST	0.32		
1966	9	HMAC Layer	HMAC	4.00		
1966	9	Original Surface	HMAC	1.50		

SID-Highway	1690 US 77
Cont.-Sect.	102 2
District	16
County	178 Nueces

Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1973	8	HMAC Layer	HMAC	2.75		
1973	8	Overlay	HMAC	1.25		

SID-Highway	3210 SH 36
Cont.-Sect.	186 2
District	17
County	26 Burleson

Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1971	5	Original Surface	HMAC	1.00		
1982	5	HMAC Layer	HMAC	2.50		
1982	5	Overlay	HMAC	1.00		
1988	8	Overlay	HMAC	1.50		

SID-Highway	3236 US 290
Cont.-Sect.	114 9
District	17
County	239 Washington

Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1969	9	Original Surface	HMAC	1.00		
1988	8	Overlay	HMAC	1.50		
1988	8	HMAC Layer	HMAC	5.38		

SID-Highway	1865 US 287
Cont.-Sect.	172 8
District	18
County	71 Ellis

Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1951	11	HMAC Layer	HMAC	1.50	7	
1957	12	HMAC Layer	HMAC	1.35	10	
1959	8	HMAC Layer	HMAC	1.60	11	
1970	4	Original Surface	HMAC	1.60	16	
1970	4	Original Surface	HMAC	1.60	17	

SID-Highway	1894 IH 30
Cont.-Sect.	9 12
District	18
County	199 Rockwall

Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1958	7	HMAC Layer	HMAC	1.50	16	
1966	10	HMAC Layer	HMAC	1.60	29	
1972	5	Original Surface	HMAC	1.19	37	

SID-Highway	2002 IH 10
Cont.-Sect.	508 2
District	20
County	36 Chambers

Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1967	10	HMAC Layer	HMAC	1.00		
1975	9	Porous Fr. Course	Open Gr.Fr.Course	1.00		
1981	5	Overlay	HMAC	1.50		
1986	9	HMAC Layer	HMAC	5.00		
1986	9	Overlay	HMAC	1.00		

SID-Highway	2324 IH 10					
Cont.-Sect.	2121 4					
District	24					
County	72 El Paso					
Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1960	7	HMAC Layer	HMAC	2.00		
1960	7	HMAC Layer	HMAC	1.50		
1969	4	HMAC Layer	HMAC	0.90		
1969	4	HMAC Layer	HMAC	2.00		
1969	4	Original Surface	HMAC	1.10		
1983	9	Part. Milled Sur.	HMAC	0.10		
1983	9	HMAC Layer	HMAC	1.00		

SID-Highway	2468 US 287					
Cont.-Sect.	42 8					
District	25					
County	65 Donley					
Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1958	6	HMAC Layer	HMAC	2.50		

SID-Highway	2879 US 287					
Cont.-Sect.	43 4					
District	25					
County	100 Hardeman					
Year	Month	Layer Description	Layer Material Classification	Center Thickness	Job No.	Coarse Aggregate and Source
1965	8	HMAC Layer	HMAC	3.00		
1965	8	HMAC Layer	HMAC	1.50		
1980	8	Overlay	HMAC	2.00		

CHAPTER 3

FIELD AND LABORATORY INVESTIGATIONS OF ASPHALT CONCRETE PAVEMENT

3.1 FIELD STUDY OF ASPHALT PAVEMENTS

A field investigation was performed under Study 1121 to provide some insight into the contributing factors of rutting of the surface layer. The results of Study 1121 with respect to the performance of the surface layer are summarized here to provide a basis for additional work under this study to investigate rutting factors in the surface layer which can be contributed to coarse aggregate type. Field sites were confined to the State of Texas and limited to pavements that were less than two years old (with one exception) and had experienced rutting depths greater than 0.4 inches. Pavements in Districts 4, 8, 10, 11 and 17 were selected for investigation (Figure 3.1). Pavements were selected for investigation only if rutting appeared to be occurring in the asphalt concrete surface layer. A visual condition survey of each pavement was conducted and rut depths were measured. A summary of the test pavements is given in Table 3.1. Environmental data are included in Table 3.2.

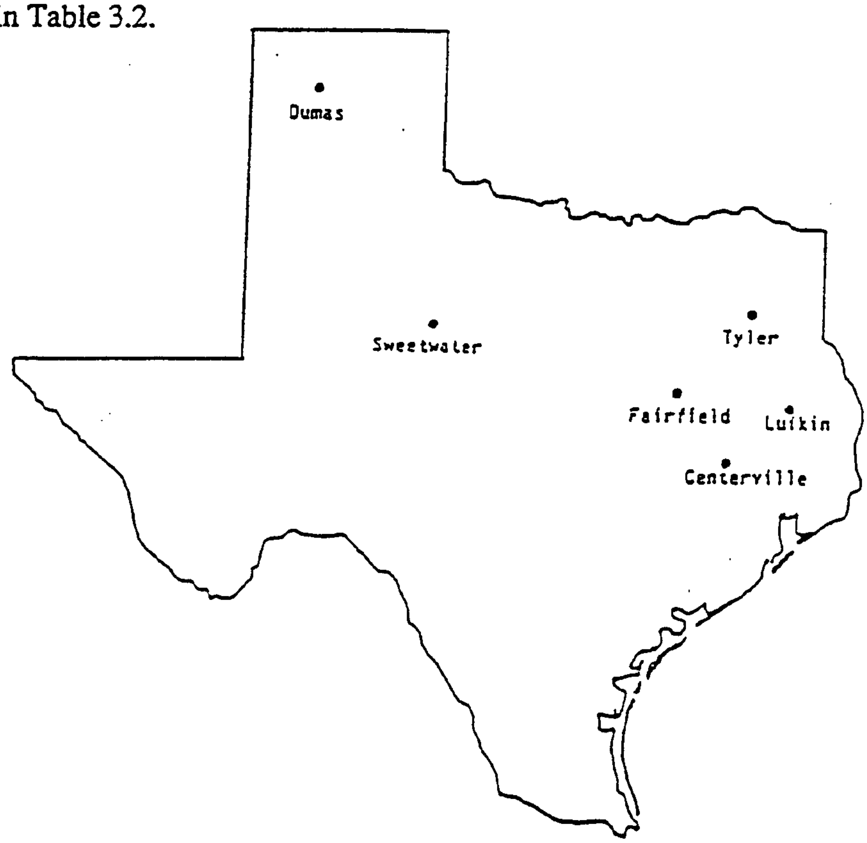

Figure 3.1: Location of field test sites for rutting pavements.

	Location					
	Sweetwater	Fairfield	Centerville	Tyler	Lufkin	Dumas
District No.	8	17	17	10	11	4
Highway No.	IH 20	IH 45	IH 45	IH 20	US 59	US 287
Existing Pavement						
Layer 1 (Top)	2½" Ty D	¾" Ty D	¾" Ty D	1½" Ty D	3" Ty D	
Layer 2	8½" Recycle	3¾" Ty C	4½" Ty C	2" Ty B	Surf Trt.	
Layer 3	Lime Trt Base	Asp. Rub.	Asp. Rub.	Fabric	Conc. Pvt.	
Layer 4	Subgrade	8" CRCP	8" CRCP	8" CRCP	Subgrade	
Date of last const.	Sept. 84	Sept. 85	Oct. 85	July 81	Nov. 85	July 85
Date Cored	March 87	April 87	April 87	Sept. 87	Dec. 87	Nov. 86
Rut Depth, in. (site 1)	0.72	0.22	0.55	0.73	0.75	0.41
Rut Depth, in. (site 2)	0.21	0.52	0.16	--	--	--

Table 3.1: Summary of rutting pavements evaluated.

Item	Sweetwater	Fairfield/ Centerville	Dumas	Lufkin	Tyler
Climate	Semiarid, mild winter, lower humidity and hot summers	Subtropical with mild winters and hot humid summers	Short but severe winters, warm summer days, cool nights, low humidity	Humid, mild winters and hot summers	Humid, mild winters and hot summers
Temperature (°F)					
Mean*/Record Max	95/111	96/111	92/109	94/108	94/108
Mean*/Record Min	31/-9	35/-3	19/-18	38/-2	33/2
Number Days/Year 20° and below	96	101	72	103	83
Frost Penetration, in.	3	<3	>3	<3	<3
Precipitation (in.)					
Mean Annual Precip.	23	39	19	42	43
Mean Annual Ice/Snow	0.8	0.8	16.2	0.8	1.9
Mean Heating *days	2620	2150	3750	1930	2330

*Mean daily minimum and maximum temperatures for a given month.

Table 3.2: Climatological summary for rutting pavement test sections.

3.2 SAMPLING AND TESTING PROGRAM

Five cores distributed across the pavement in and between the wheelpaths (Figure 3.2) were drilled in order to ascertain the profile for the transverse cross section of the pavement. Cores were drilled in accordance with this scheme at each of five different locations at any particular field site. The surface layers of the cores were carefully separated in the laboratory by sawing and later tested.

In some instances, the surface layer of the asphalt concrete pavement was found to be less than 1-inch thick. In this instances, the top surface layer was tested for air void content, asphalt content, asphalt viscosity, aggregate gradation, and aggregate classification. Otherwise the tests shown in Figure 3.3 were conducted in the laboratory.

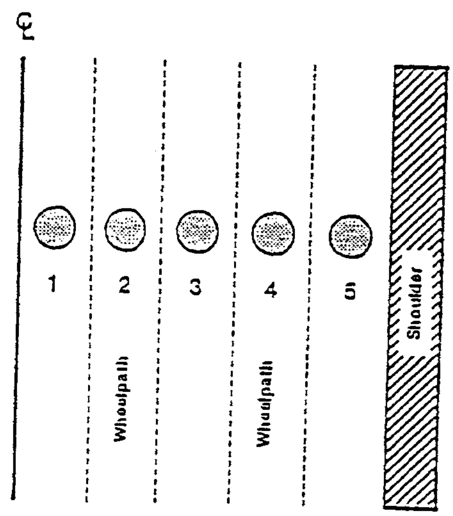

Figure 3.2: Distribution of cores across the pavement.

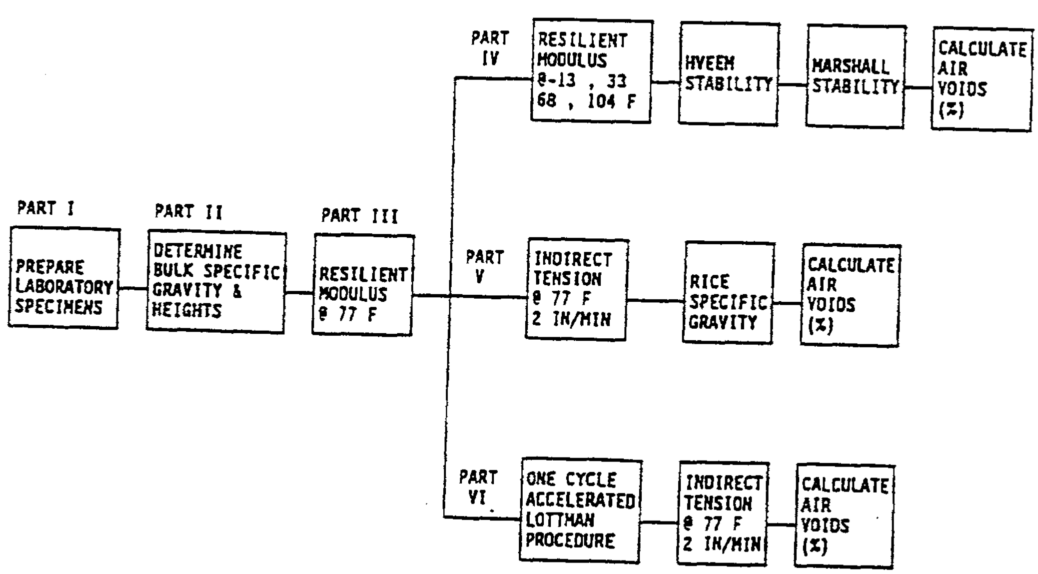

Figure 3.3: Laboratory test program.

3.3 TEST RESULTS

Laboratory test results are described in the following subsections for each of the pavement sites analyzed. Test results were separated by location. Mixture properties were analyzed and compared.

Sweetwater. In Sweetwater, the asphalt pavement at site 1 exhibited more premature rutting (Table 1) than the pavement at site 2 primarily because the average asphalt content extracted from the cores in site 1 was much higher (0.7 percent more) than the average asphalt content found in the cores from site 2 (Table 3.3). With the interaction of these three major factors, a considerable increase in rutting susceptibility occurred.

	Sweetwater		Fairfield		Centerville		Tyler	Lufkin	Dumas
Site No.	1	2	1	2	1	2	1	1	1
Penetration									
77°F,100gm,5sec	37	36	27	44	27	36	72	56	65
39.2°F,200gm,60sec	10	11	13	15	5	3	-	21	19
Viscosity, poise									
140°F	2230	2330	10,710	5,170	6,150	4,210	2,520	4,170	1,800
275°F	3.20	3.30	5.63	3.61	5.05	4.26	-	4.90	5.39
Asphalt Content, Percent	5.3	4.6	5.3	4.7	5.6	5.0	8.7	9.5	7.0
Design Asphalt Content	5.0	5.0	4.9	4.9	5.1	5.1	8.1	8.5	Unknown

Table 3.3: Data for asphalts extracted from pavement cores.

Another important factor to consider is that the aggregate gradation curves, for both sites, are very close to the maximum density line. This means that the gradations are very dense and, therefore, very sensitive to asphalt content. Mixtures with this type of aggregate gradation may become unstable with a slight excess of asphalt.

Although Hveem and Marshall stability yielded higher average values for site 2 (Table 3.4), other material properties like VMA and resilient modulus were quite similar for both sites. Air voids were found to be considerable low (Table 3.5) for both sites. Although the sand content was quite low (12 percent), the sand particles were found to be very rounded, smooth, and nonporous, for both sites (Table 3.6).

Fairfield. The problems noted at this test site seemed to be largely related to the asphalt base rather than the surface. It is important to note that, in this case, the surface layer could not be meaningfully tested in the laboratory because it was less than one inch thick.

Location	VMA, percent[1]	Resilient Modulus, psi x 10^3 13°F[2]	33°F[2]	68°F[2]	77°F[1]	104°F[2]	Hveem Slab[2]	Marshall Stab, lbs[2]	Marshall Flow, 0.01"[2]
Sweetwater - 1	13.6[3]	1850	1396	489	344	37	8	650	17
Sweetwater - 2[4]	12.8[3]	2015	13.64	601	551	63	20	850	15
Fairfield - 1[4]	18.9	2110	1540	930	910	250	45	1450	16
Fairfield - 2	15.2	1940	1330	780	750	230	36	1500	16
Centerville - 1	16.1	2080	1650	804	560	84	44	3000	11
Centerville - 2[4]	14.5	1880	1650	880	680	140	44	2700	13
Tyler	22.1	1430	900	420	300	57	44	2600	13
Lufkin	16.0	1490	860	230	170	23	32	960	11
Dumas	22.0[3]	1600	1060	360	250	35	24	1900	16

[1]Average of 25 values (in wheelpath and outside wheelpath)
[2]Average of 6 values (3 in wheelpath, 3 outside wheelpath)
[3]Based on estimated value of bulk specific gravity of aggregate of 2.65
[4]Less rutted than other site near same location

Table 3.4: Mixture properties of pavement cores.

Pavement Location	Air Voids Content, percent					
	Adjacent to Centerline	Wheelpath	Between Wheelpath	Wheelpath	Adjacent to Shoulder	Average
Sweetwater - site 1	2.5	1.6	1.7	1.2	1.5	1.7
Sweetwater - site 2	1.8	0.9	3.1	0.9	1.5	1.6
Fairfield - site 1	8.1	6.5	7.5	6.6	6.8	7.1
Fairfield - site 2	4.8	4.1	5.7	5.4	5.8	5.2
Centerville - site 1	2.4	1.3	1.2	1.0	1.9	1.6
Centerville - site 2	3.9	1.1	1.0	2.1	2.0	2.0
Tyler	2.1	2.8	2.7	2.9	2.7	2.6
Lufkin	3.3	2.8	3.7	3.5	4.0	3.5
Dumas	9.9	4.1	6.9	4.7	6.5	6.4

Table 3.5: Summary of air voids in and outside wheelpaths.

Pavement Location	Aggregate Type	Aggregate Blend	Particle Shape +#40	Particle Shape -#40	Particle Texture +#40	Particle Texture -#40	Porosity +#40	Porosity -#40
Sweetwater (surface)	• Limestone - 59% Martin Source • Screenings - 29% Martin Source • Sand - 12% Strain & Sons	62% -[illegible]+#4, 30% -#4+#10 13% -#4+#10, 49% -#10+#40 24% -#80+#200, 14% -#200 5% -#10+#40, 62% -#40+#80 29% -#80+#200, 4% -#200	Angular to Subangular	Angular to Subangular	Rough to Smooth	Smooth	Porous	Nonporous
Fairfield (surface)	• Type D Rock - 60% O. P. Frost • Screenings - 30% O. P. Frost • Field Sand - 10% Bonner Field	97% +#10 14% +#10, 40% -#40 100% -#40	--	--	--	--	--	--
Centerville (surface)	• Type D Rock - 74% Crushed Sandstone • Fines - 26% East Texas Stone	87% +#10 3% +#10, 33% -#40	--	--	--	--	--	--
Tyler	Surface • Lightweight - 50% (TXI Streetman) • Concrete Sand - 15% (Horton Sand & Gravel) • Field Sand - 18% (J. Fair - local) • Field Sand - 17% (Lone Star Gas)		Subrounded to Rounded	Angular to Subangular	Rough to Smooth	Smooth	Porous	Nonporous
Lufkin	• Lightweight Aggregate • G & I Sand - 27% • Elliot Sand - 11%	71% -5/8"+#4, 24% -#4+#10 13% -#10+#40, 68% -#40+#80 70% -#40+#80, 17% -#80 +#200, 10% -#200	Subangular	Angular	Rough	Smooth	Porous	Porous and Nonporous
Dumas	--	--	Subangular	Subangular	Rough	Smooth	Porous	Nonporous

Table 3.6: Aggregate gradations and physical characteristics, all sites analyzed.

Centerville. In Centerville, the asphalt pavement at site 1 exhibited more rutting (Table 3.1) than the pavement at site 2. This can be explained by the interaction of the following factors:

1. The average asphalt content extracted from the cores of site 1 is higher (0.6 percent more) than the average asphalt content obtained from the cores of site 2 (Table 3.3).

2. The sand-sized particles found at both sites are rounded,-smooth, and nonporous (Table 3.4) making the mix sensitive to binder content.

3. The gradations are on the fine side of the specification and exhibit a large hump in the curve at the number 40 sieve for both sites, indicating a critical mixture that becomes readily unstable with a slight excess of fluids.

Another factor to mention is that the air void profile (Table 3.6) indicates that the Centerville pavement was probably compacted below the normally specified levels during construction. Thus, the higher binder content of site 1 is the critical factor differentiating the performance between the two sites.

Tyler. This pavement was placed as a fabric test section in 1981 at a very high-traffic area of IH 20. Rutting leveled off for four years and then, in the spring of 1987, it began to increase dramatically (Figure 3.4). The surface required milling and overlaying by July of 1987.

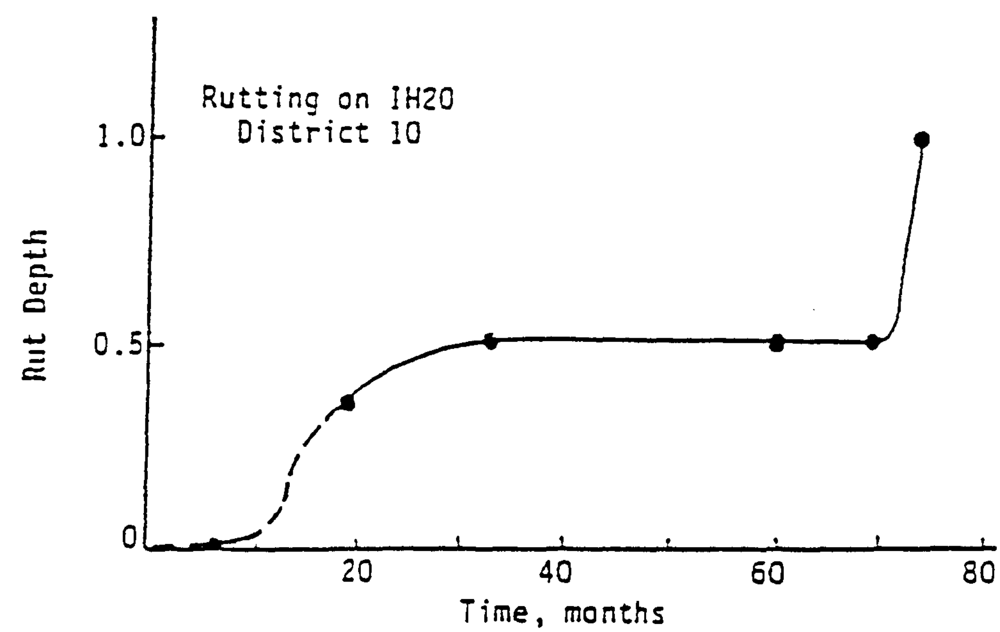

Figure 3.4: Rutting history of pavement on IH20 near Tyler, Texas.

It seems that the most important factor that contributed to rutting in this particular site was the amount and character of the natural aggregate. The surface mix contained 50 percent natural sand and gravel composed of rounded, smooth, and nonporous aggregate particles. The surface mix was also gap graded, exhibiting a notable hump in the gradation curve at the number 40 sieve. Both the gradation and character of the aggregate particles determined the performance of the mixture.

It was also found that the surface mix contained 0.6 percent more asphalt, on the average, than the design asphalt content (Table 3.3).

Based on laboratory tests, the surface mixtures indicated severe moisture susceptibility (Table 3.7); however, visual examination of the cores did not indicate that moisture had damaged the surface layer.

Location	Before Moisture Treatment				After Moisture Treatment				
		Tensile Properties[1]				Tensile Properties[1]			
	Average Air Void Content (%)	Tensile Strength (psi)	Strain @ Failure (in/in)	Secant Modulus (psi)	Average Air Void Content (%)	Tensile Strength (psi)	Strain @ Failure (in/in)	Secant Modulus (psi)	TSR[2]
Sweetwater - 1	1.7	142	0.0086	78,000	1.9	151	0.0013	82,000	106
Sweetwater - 2	1.2	175	0.0032	69,000	2.9	160	0.0023	64,000	91
Fairfield - 1	8.4	200	0.0015	154,000	6.3	174	0.0017	103,000	87
Fairfield - 2	4.8	188	0.0013	147,000	5.9	116	0.0045	51,000	62
Centerville - 1	2.8	268	0.0028	97,000	0.5	275	0.0031	92,000	103
Centerville - 2	1.3	289	0.0025	132,000	1.1	181	0.0022	86,000	63
Tyler	3.1	175	0.0024	75,000	3.4	95	0.0050	19,000	54
Lufkin	2.2	119	0.0040	30,000	4.5	74	0.0044	18,000	62
Dumas	4.7	143	0.0017	58,000	9.9	74	0.0042	18,000	52

[1]Tensile tests were performed at 77°F and 2 inches per minute (Average values are reported).
[2]Tensile strength ration = (Tensile strength after / Tensile strength before) x 100.

Table 3.7: Tensile properties of cores before and after Lottman freeze-thaw moisture treatment.

Lufkin. The Lufkin mix contained acceptable air voids (Table 3.6) and voids in the mineral aggregate (VMA) above the minimum percentage (15 percent) recommended by the Asphalt Institute (Table 3.5). However, this mix contained 37 percent by weight natural sand particles (Table 3.5) with an asphalt content one percent above optimum design (Table 3.3) and, as a result) exhibited relatively low stability. the coarse material in this mixture was lightweight synthetic aggregate. The aggregate grading exhibited excessive amounts (Item 340, Type D) of minus number 40 sieve size material. This may be due partially to aggregate degradation during plant operations, pavement service, and coring. The mixture also exhibited sensitivity to moisture. This combination of factors in the presence of the incessant traffic of US 59 resulted in severe rutting after two summers. This pavement was milled off in the spring of 1988 to remove ruts near 1-inch deep.

Dumas. Tests on the Dumas cores showed relatively high air voids and a mixture of low stability and poor resistance to moisture damage. Table 3.6 shows significantly lower air voids in the wheelpaths than between or outside the wheelpaths indicating inadequate compaction during construction. Visual examination of the cores indicated stripping of asphalt from the large and intermediate size aggregate. Inadequate compaction may have provided permeable voids which enhanced stripping, which in turn contributed to rutting. The design asphalt content is not known, but the actual content appears quite high. Another factor contributing to the rutting problem could be the low viscosity of the asphalt.

3.4 SUMMARY OF FINDINGS

Based on the findings from the field investigation, the following factors are believed to be the primary contributors to rutting:

1. All the aggregate systems analyzed were found to be dense graded and contained from 12 percent to 50 percent natural, rounded, nonporous particles (most of which are sand-sized particles). This aggregate system may exacerbate rutting as it provides little room for asphalt and is quite sensitive to asphalt content

2. Binder content in the asphalt-aggregate mixture was in excess of that required by the optimum mixture design.

3. Heavy traffic enhanced rutting when the previously discussed factors were present in an asphalt mixture.

The combination of these primary contributors notable accelerates the rutting process.

In Study 1121, the focus concentrated on the influence of the amount and character of the sand-sized aggregate particles on permanent deformation which was found to be of great significance in the field investigation. Several laboratory testing methodologies were used to estimate relative rutting potential of mixtures containing various percentages of natural sand.

CHAPTER 4

ANALYSIS OF AC BEHAVIOR AND AC PAVEMENT DISTRESSES

This chapter addresses the fundamental analysis to address the specific pavement distress studied in this project and the associated analysis to describe the characteristics of each in terms of significant material properties. This analysis forms the basis by which to predict pavement performance and improved design procedures. Asphalt pavement rutting is considered first and PCC pavement cracking and spalling is addressed subsequently.

4.1 DESCRIPTION OF PERMANENT DEFORMATION CHARACTERISTICS OF ASPHALT MATERIALS

The loading-unloading response (0-E) curve of pavement materials is shown schematically in Figure 4.1. The strain is decomposed into two components, E_e - the elastic (resilient) and E_p the permanent (residual) strain. The total strain E_t is the same $E_e + E_p$. The resilient strain remains fairly constant during the major part of the life except, at low number of repetitions (the material undergoes conditioning) and near failure. In the analysis, it is assumed that the elastic strain is constant throughout the pavement life. The residual strain per load application (ΔE_p) is shown to decrease with number of load applications. To follow an exponential law which is linear on the log-log scale in Figure 4.2, where N is the number of repetitions, I is the intercept on the permanent strain axis, and s is the slope of the curve. The equation for the permanent strain is:

$$\epsilon_p = \int d\epsilon_p = IN^s \tag{1}$$

then:

$$\Delta\epsilon_p^{(N)} = \frac{\partial \epsilon_p}{\partial N} = ISN^{s-1} \tag{2}$$

defining:

$$\alpha = 1 - s \qquad \mu = \frac{IS}{\epsilon_e} \tag{3}$$

provides:

$$\Delta\epsilon_p^{(N)} = \epsilon_e\, \mu N^{-\alpha} \tag{4}$$

When:

$$N = 1 \qquad\qquad \mu = \frac{\epsilon_p^{(1)}}{\epsilon_e} \; S$$

From Figure 4.1 it is seen that:

$$E_{l} = \frac{\sigma}{\epsilon_t(N)} \quad = \text{ modulus during loading}$$

$$E_{un} = \frac{\sigma}{\epsilon_e} \quad = \text{ modulus during unloading}$$

Knowing E_{un}, along with a and μ of the material, one can write:

$$E_{l} = \frac{\sigma}{\Delta\epsilon_p^{(N)} + \epsilon_e} = \frac{\sigma}{\epsilon_e(\mu N^{-\alpha} + 1)} = \frac{E_{un}}{1 + \mu N^{-\alpha}} \tag{5}$$

This equation provides a relation between the (varying) modulus during loading and the (constant) modulus during unloading as a function of alpha (a), and gnu (μ), the permanent deformation characteristics. This equation is used in the simulation program.

Gnu is defined as the ratio between permanent strain at N = 1 and the elastic strain. Since the permanent strain at N = 1 is very difficult to measure accurately, the variability of μ measured in the laboratory may be unrealistically large.

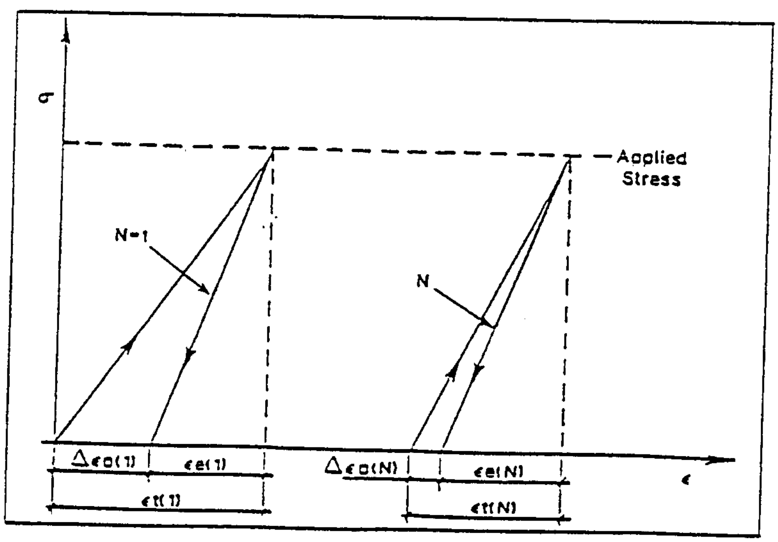

Figure 4.1: A typical loading / unloading response for a pavement material.

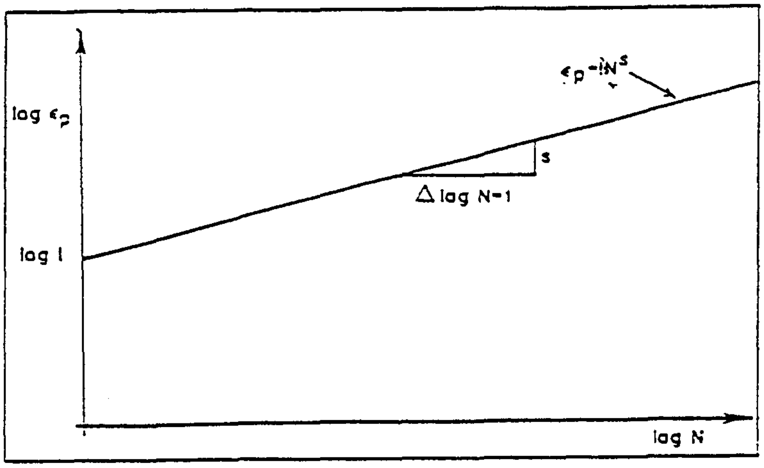

Figure 4.2: Pavement Deformation Characterization.

4.2 DEVELOPMENT OF A NEW RUTTING MODEL

A novel theoretical approach, founded on the creep and recovery behavior of asphalt concrete mixtures, was developed into a rutting model. The resulting hyperbolic equations accurately model the deformation performance of a paving mixture.

It has been found that both creep and recovery compliance behaviors of an asphalt mixture follows curves similar to the ones shown in Figures 4.3 and 4.4. Using a philosophical approach developed by Badillo (Ref. 4.1), an equation for predicting the creep compliance behavior of an asphalt concrete mixture can be derived:

$$D(t) = \frac{D_o + D_m a t^m}{1 + a t^m} \tag{6}$$

where

D_o	=	initial creep compliance,
D_m	=	maximum creep compliance,
a	=	regression constant,
t	=	time, and
m	=	slope factor.

The same philosophical approach can be applied to the recovery compliance, even though it shows a different slope factor and a partial strain recovery. The equation obtained for this recovery compliance is represented as follows:

$$R(t) = \frac{R_o + R_m bt^{mp}}{1 + bt^{mp}} \tag{7}$$

where

R_o	=	initial (elastic) recovery compliance,
R_{max}	=	maximum recovery compliance,
b	=	regression constant,
t	=	time, and
p	=	slope modifier factor.

In order to obtain all the unknown parameters in Equations 1 (D_o, D_{max}, a, m) and 2 (R_o, R_{max}, b, p), an optimization technique known as pattern search is used. Pattern search is a technique based on an optimization method developed by Hooke and Jeeves (Ref. 4.2). The method consists, basically, of finding the unknown parameters that minimize the sum of the squared differences between observed data and predicted values.

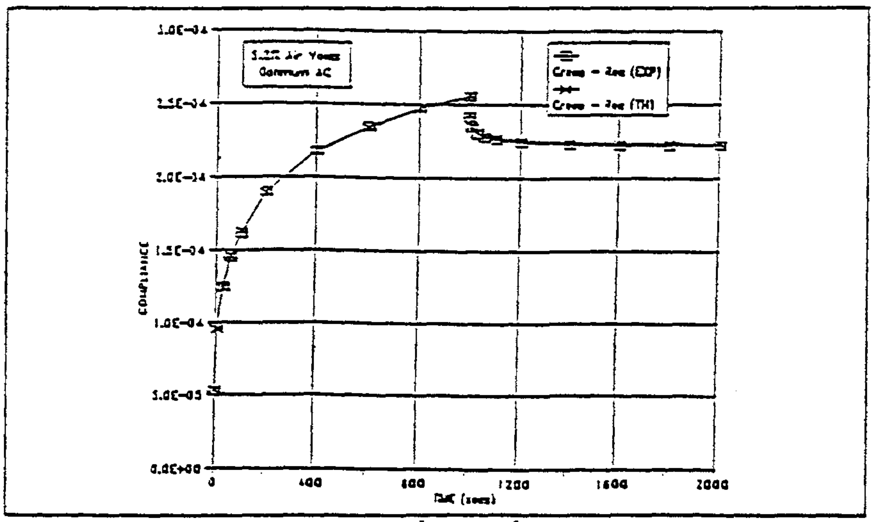

Figure 4.3: Linear representation of typical creep and recovery behavior of a 0% natural sand mix at high air void contents.

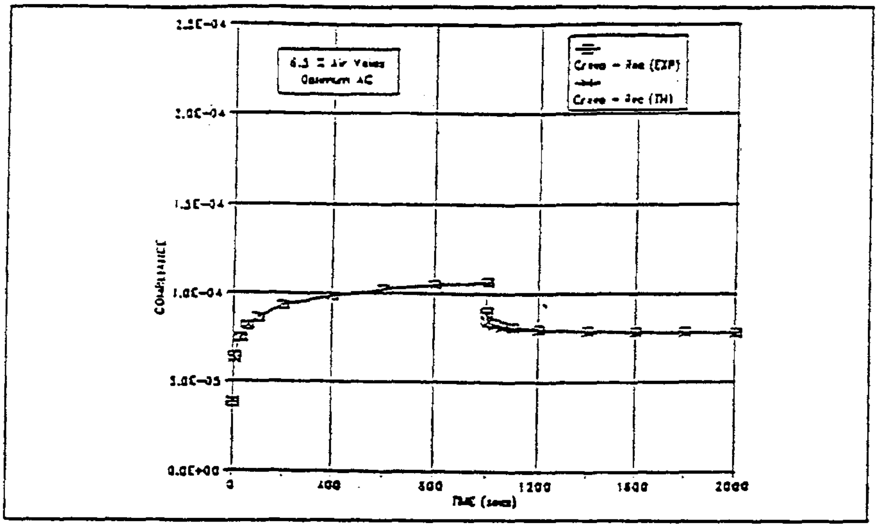

Figure 4.4: Linear representation of typical creep and recovery behavior of a 40% natural sand mix at high air void contents.

4.3 Test Results

Figures 4.3 and 4.4 represent typical results from the laboratory test program for mixtures with low and high susceptibility to rutting, respectively. The mix considered highly susceptible to rutting is the 40 percent natural sand mix, while the mix considered to be rut-resistant is the zero percent natural sand mix. Both figures include experimental results, as well as theoretical predictions obtained by using the hyperbolic models described above.

The results obtained from all the cases analyzed reveal that:

1. The hyperbolic modeling of the creep and recovery behavior produced near-perfect predictions.
2. The value of "p" defined for Equation 2 increases as the percentage of natural sand in the asphalt mixture increases (Table 4.1).

Based on these results, one can infer that the "p" value could account for the role of the natural sand in the permanent deformation behavior of asphalt concrete mixtures. This constitutes a novel way of characterizing the influence of the aggregate's physical nature in a permanent deformation model. More research is needed to analyze the sensitivity of the "p" value to variations in asphalt binder characteristics and to changes in the type and character of the coarse aggregate used in the mixture.

Table 4.1: Mean "p" values at high air void contents.

Mix type	Mean "p" value for 3 tests
0 % natural sand	0.8
20 % natural sand	1.4
40 % natural sand	1.8

4.4 Permanent Deformation Model

A permanent deformation model was derived from Equations 6 and 7. Suppose that a load pulse of duration Δt is applied repeatedly, N times. Then, compliance equations for both creep and recovery may be used to estimate the total deformation and recovery that occurs:

$$D(N) = \frac{D_o + D_m r N^m}{1 + rN^m} \tag{8}$$

$$R(N) = \frac{R_o + R_m r_p N^{mp}}{1 + r_p N^{mp}} \tag{9}$$

where

N = number of cycles,
r = $a\,(\Delta t)^m\,\rho$,
r_p = $b\,(\Delta t)^{mp}\,\rho_p$,
t = $N\,(\Delta t)$, and
ρ, ρ_p = load pulse factors.

When the load is not a square wave, the term $(\Delta t)^m$ must be multiplied by p, which is a function of m and the loading wave shape, and varies between 0 and 1. For a square wave, p is equal to 1. The factor, p_p, for the recovery curve has the same meaning as p, and it is a function of p, mp, and the wave shape. For a square wave, p_p is also equal to 1.

The total accumulated strain after N repetitions for a constant stress pulse, s_o, is defined as:

$$\epsilon_a(N) = \epsilon(N) - \epsilon_{re}(N) = \sigma_o[D(N) - R(N)] \tag{10}$$

The rate of change of the permanent strain with load repetitions is:

$$\frac{\partial \epsilon_a(N)}{\partial N} = \sigma_o \left[\frac{\partial D(N)}{\partial N} - \frac{\partial R(N)}{\partial N}\right] \tag{11}$$

Replacing equations (8) and (9) in equation (10), and dividing by the resilient strain, e_r, gives:

$$\frac{1}{\epsilon_r}\frac{\partial \epsilon_a(N)}{\partial N} = E_r D_m m N^{m-1}\, q(1 - qN^m)\ [(D_m - D_o) - (R_m - R_o)ps] \tag{12}$$

where

$$q = \frac{r}{1 + rN^m} \tag{13}$$

$$s = \frac{q_p(1 - q_p N^m)}{q(1 - qN^m)} \tag{14}$$

$$q_p = \frac{r_p N^{m(p-1)}}{1 + r_p N^{mp}}$$

E_r = resilient modulus (15)

Now, recalling the permanent strain response model used in the VESEY approach (Ref. 4.3).

$$\frac{\partial \epsilon_p}{\partial N} = \epsilon_r \cdot \mu \cdot N^{-\alpha} \tag{16}$$

where

μ, α = parameters determined from equations 12 and 13,
N = number of cycles,
ϵ_r = elastic or resilient strain, and
$\frac{\partial \epsilon_p}{\partial N}$ = rate of change of permanent strain with load repetitions.

Thus, if

$$\mu N^{-\alpha} = \frac{1}{\epsilon_r}\left[\frac{\partial \epsilon_a}{\partial N}\right]$$

then, it is apparent from equation 12 that

$$\alpha = 1-m \tag{17}$$

$$\mu = E_r D_m mq \;(1 - qN^m)\; [(D_m - D_a) - (R_m - R_a)ps] \tag{18}$$

where

$$s = \frac{q_p(1 - q_p N^m)}{q(1 - qN^m)} \tag{19}$$

The μ and a parameters defined in equations 17 and 18 can be easily incorporated into the Texas Flexible Pavement System (TFPS) program (Ref. 4.4) developed at the Texas Transportation Institute in order to predict rutting. In this approach, the loading-unloading response (s - e curve) of the pavement material is modeled as in Figure 4.1. The strain response is decomposed into E_e-elastic (resilient) strain, and E_p-permanent strain. The total strain, E_t, is the sum of $E_e + E_p$. The elastic (resilient) strain remains fairly constant during the life of the pavement except at low number of load repetitions and near failure. The change in permanent strain per load application (ΔE_p) decreases with the number of load applications until the sample reaches failure, where it starts to increase dramatically (Ref. 4.5). In general, the permanent strain is represented by equation (10).

From Figure 4.1 it is observed that

$$E_{la}(N) = \frac{\sigma}{\epsilon_t(N)} \quad = \text{modulus during loading} \tag{20}$$

$$E_{un} = \frac{\sigma}{\epsilon_a} \quad = \text{modulus during unloading} \tag{21}$$

Now, rewriting Equation 15 by using Equation 11 gives:

$$E_{lo}(N) = \frac{\sigma}{\Delta\epsilon_p(N) + \epsilon_r} = \frac{\sigma}{\frac{\partial\epsilon_p}{\partial N} + \epsilon_r} = \frac{\sigma}{\epsilon_r(1 + \mu N^{-\alpha})} = \frac{E_{un}}{1 + \mu N^{-\alpha}} \tag{22}$$

This equation provides a relation between $E_{lo}(N)$ and E_{un} (constant), as a function of a, μ, and N. The permanent deformation under load is then calculated by subtracting the surface rebound, calculated while all the pavement layers are assigned the unloading moduli (E_{un}), from the surface deflection, calculated while all pavement layers are assigned the loading moduli (E_{lo}).

The influence of rounded sand in the laboratory mix is can be illustrated by this rutting model (Ref. 4.5) in that the loading moduli,(E_{lo}), are 1.8 times greater for the 0% natural sand mix than for the 40% natural sand mix. This indicates a much lower rutting potential for the 0% sand mix at a given level of traffic and temperature.

The hyperbolic model provides a near perfect predictor of actual creep and recovery performance of asphalt mixtures. The "p" value defined within the hyperbolic model shows a strong correlation with the percentage of natural sand in a mix, hence it accounts for the influence that the character of the aggregate particles has on deformation behavior. The hyperbolic models for creep and recovery can be mathematically transformed, as shown in previous pages, into a permanent deformation prediction model. Additional work is needed to perform a complete calibration of the new model.

4.5 DESCRIPTION OF CALCULATION ROUTINE

The surface deflection under load is calculated, while all pavement layers are assigned the loading moduli, E_{lo}, and the surface rebound during unloading is subtracted while all pavement layers are assigned the unloading moduli, E_{un}. The incremental residual deflection is the difference between the two deflections (Figure 4.5). Since the E_{un} are assumed constant, the unloading surface deflection is computed only once. The process is repeated for different N-values, to determine values of the rate of rutting as function of N.

Since rut depth is defined as the depth of the depression in the wheel path with reference to a 4-ft. straight edge, and the transversal distribution is normal with an assumed standard deviation of one foot, the computations must be made at both the wheel path center line and 2-ft. apart and for different positions of the dual wheel from the wheel path centerline (Figure 4.6).

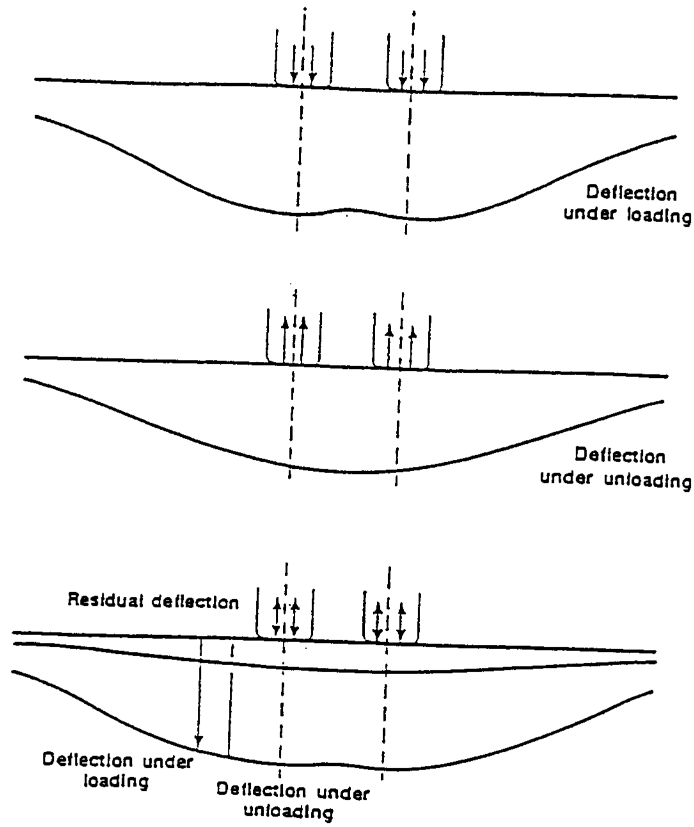

Figure 4.5: Schematic description of rutting development.

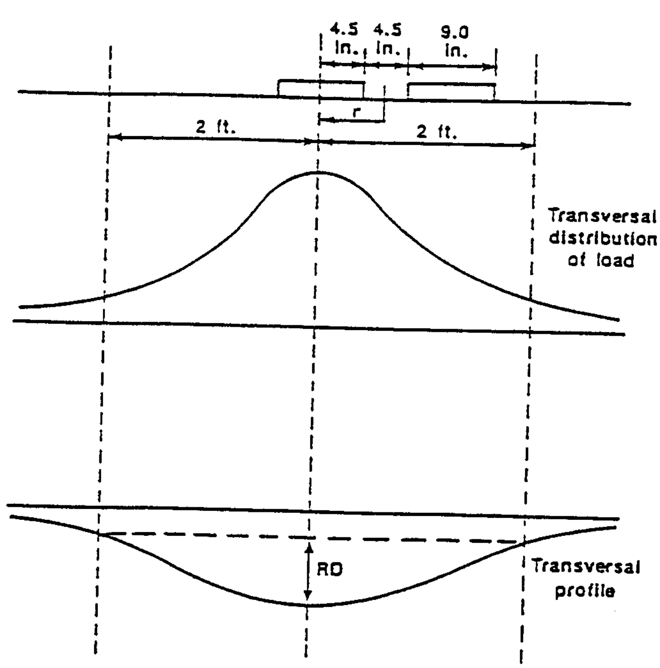

Figure 4.6: Transverse distribution of load and surface deformation.

To account for all of these factors, an equation was derived to compute the incremental residual deformation RR from the surface deflection at different offsets:

$$RR = 0.118\Delta\delta_1 + 0.3224\Delta\delta_2 - 0.013\Delta\delta_3 - 0.246\Delta\delta_4 - 0.181\Delta\delta_5 \qquad (23)$$

where:

RR = The incremental rut depth for the given transversal distribution of load and measured with a 4 - ft. straight edge.

Δd_i = the difference of the surface deflections under loading and unloading,

i = 1 corresponds to r/a = 1.17

i = 2 corresponds to r/a = 1.50

i = 3 corresponds to r/a = 4.00

i = 4 corresponds to r/a = 6.70

i = 5 corresponds to r/a = 9.40

where:

a = radius of contact area

Since a large amount of data is required to cover the wide range of plastic properties of the pavement material, a separate program (see Appendix A) was written to compute RR at N = 1, 100 and 10,000 load repetitions and to derive a_1 and a_2 in the assumed shape of RR equation from equation (23):

$$RR = \frac{pa}{E_{sg}} a_1 N^{a_2} \qquad (24)$$

where:

p = tire pressure

E_{sg} = modulus of the subgrade

A set of surface deflections at r/a = 1.17, 1.5, 4.0, 6.7 and 9.4 was generated using the BISAR computer program for the following conditions:

$$\frac{T_1}{a} = 0.4,\ 1.1,\ 2.2$$

$$\frac{T_2}{a} = 0.5,\ 1.3,\ 2.6$$

$$\frac{T_3}{a} = 0,\ 1.3,\ 2.6,\ 5.2$$

$$\frac{E_1}{E_{sg}} = 10,\ 25,\ 60,\ 150,\ 350$$

$$\frac{E_2}{E_{sg}} = 2,\ 5,\ 10,\ 25,\ 60,\ 150,\ 350$$

$$\frac{E_3}{E_{sg}} = 1,\ 2,\ 5,\ 10$$

The dataset was used with interpolation techniques to compute equation (23). The procedure follows as:

1) Read data which includes pavement layer thicknesses, resilient moduli and plastic properties (a's and μ's of layers)

2) Compute unloading surface deflections at five radial offsets.

3) Set N = 1; compute loading moduli using E_{un}, a, μ for each layer .

4) Compute loading surface deflections at five radial offsets, using the dataset and interpolation techniques.

5) Compute RR from equation for N = 1.

6) Repeat steps 3) to 5) for N = 100 and 10,000.

7) Compute a_1 and a_2 by regression analysis.

Several thousand combinations of pavement layer thicknesses, moduli and plastic parameters were used to generate a_1 and a_2 coefficients. The results were used to derive a polynomial for a_1 and a_2 as a function of the surface deflection under one wheel of the dual wheel load (to represent the pavement stiffness) and of the plastic parameters.

The characterization of a and μ in the TFPS program for asphalt concrete material is described as follows. The a-parameter is assumed to depend on the stiffness, temperature, and state of stress of the asphalt layer. The parameter makes use of McLeod's nomograph (Ref. 4.6) and the tensile strain at the bottom of the layer in the following way:

$$\alpha = 1 - \frac{m_c(T)}{m_c(68°F)} \quad (0.1 + B_1 \exp\left[-\left[\frac{20}{E \cdot \epsilon_R}\right]^2\right]) \tag{25}$$

where:

$m_c(T)$ = represents the change in material stiffness (E(t))with a change in loading time (t) at temperature T,
E = represents the stiffness of the asphalt layer,
E_T = represents the tensile strains at the bottom of the asphalt layer, and
B_1 = represents a calibration constant

The equation for m_c is found for asphalt materials as a function of two different loading times, t, and t_2:

$$m_c = \frac{\mathrm{Log}\left[\frac{E(t_1)}{E(t_2)}\right]}{\mathrm{Log}\left[\frac{t_2}{t_1}\right]} \tag{26}$$

The μ parameter is set to:

$$\mu = B_2 \{P\}(1-\alpha) \tag{27}$$

where:

t = represents the loading time, and
B_2 = represent a calibration constant
P = Creep factor dependent upon the rutting resistant characteristics of the asphalt mix and quality of the aggregate (0.9 for high quality; 0.4 for low quality)

The a and μ- parameters are indirectly a function of P since they change with P as a function of m_c and provides for an affect due to the quality of the aggregate. The effect of the aggregate quality is also manifest in the E value for the asphalt surface layer. A correlation is suggested for this purpose based on the effect of the optimum asphalt content (a) of the asphalt mix since the quality of aggregate can influence the optimum asphalt content. A relation (Ref. 4.7) of the form:

$$\mathrm{Log}_{10}| E | = C_1 + C_2 \cdot (P_{ac} - a)^{0.5} \tag{28}$$

which is very similar to equation 77 shown previously. Since the expressions for C_1 and C_2 are essentially given in equation 77 and provided in reference 4.7, they are not shown here again. The only difference is the optimum asphalt content term included to correct the equation. This term is correlated to the quality of the aggregate (through P) on the following basis:

$$P = 0.9 - 0.35 \cdot a \tag{29}$$

which is valid for optimum asphalt contents between 4 and 6 percent. This relation is used to determine the parameter based on the above ranges for p in terms of the aggregate quality to adjust the layer stiffness to determine the effect on rutting development as a function of the coarse aggregate in the asphalt mixture.

4.6 REFERENCES

4.1. Juarez-Badillo, E., "General Compressibility Equation for Soils," Tenth Internatioinal Conference on Soil Mechanics and Foundation Engineering, Stockholm, Sweeden, Vol. 1, pp. 171-178.

4.2. Hooke, R. and Jeeves, T. A., "Direct Search Solution of Numeral and Statistical Problems," J. Association Comp. Mach., 8,2, April, 1961.

4.3. Kenis, W. J., "Predictive Design Procedures," VESYS User's Manual, FHWA Report 77-154, 1978.

4.4. Uzan, J. and Smith, R. E., TFPS Technical Report, Texas Transportation Institute, Texas A&M University, College Station, Texas, August, 1988.

4.5. Perdomo, D. and Button, J. W., "Identifying and Correcting Rut-Susceptible Asphalt Mixtures," Research Report 1121-2F (Draft), Texas Transportation Institute, Texas A&M University, College Station, Texas, 1991.

4.6. McLeod, N. W., "Asphalt Cements: Pen-Vis Number and its Application to Moduli of Stiffness", Journal of Testing and Evaluation, ASTM, Vol. 4, No. 4, July, 1976.

4.7. Miller, John S., Uzan, Jacob, and Witczak, Matthew W., "Modification of the Asphalt Institute Bituminous Mix Modulus Predictive Equation," TRR 911, pp. 27-35.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

Although research efforts on performance of asphalt concrete (AC) and portland cement concrete (PCC) pavements had resulted in the development of databases which provide some insight into pavement behavior characteristics, but few data about effects of different coarse aggregate types on the pavement performance were available before this research project - Project 1244, "Evaluation of the Performance of Texas Pavements made with Different Coarse Aggregates". Consequently, the design and construction of the pavements in Texas have not completely taken into account the variation in pavement material properties that may be attributed to the use of different coarse aggregate types. The goal of this project is to evaluate the performance of rigid pavements and flexible pavements made with siliceous river gravel and with crushed limestone as coarse aggregates in order to determine the extent of performance differences between these and other types of aggregates. By determining the difference of performance of these aggregates, as well as by determining the main factors that affect these aggregates when used to build pavements in Texas, one can then make different design adjustments and adaptation for pavements made with different aggregates.

Under this project, literature review and field investigation have been performed on the performance of asphalt concrete and portland cement concrete pavements up to this date. Fracture tests of PCC beam specimens made with different coarse aggregates available in Texas have been conducted in the laboratory. Experimental methods for characterizing texture, shape, and elongation of the aggregate based on the concept of fractals have been developed by using image analysis techniques. A theory on creep and recovery behavior of asphalt concrete mixtures has been reviewed and planned to be applied to tests for evaluating the effect of coarse aggregates on rutting resistance. A fracture mechanics based theoretical analysis on the pavement of portland cement concrete has been fulfilled for determining sawcut depth and spacing. These achievements have provided a solid basis for further field investigation and laboratory work. As a result, pavement test sections can be will arranged in order to observe effects of coarse aggregates, curing methods and crack control methods (conducting sawcutting, placing crack inducers, skewing reinforcing steel rebars, etc.). Work done for PCC materials and PCC pavements are summarized in Volume II of this report. Conclusions and recommendations from each part of work related to asphalt concrete and AC pavements are summarized as follows.

5.1 PAST FINDINGS ON AC PAVEMENTS

Factors affecting AC pavement performance have been closely examined and evaluated under this project. Rutting in asphalt pavement develops due to accumulated movement of materials under repeated loads, either in the pavement layers and/or in the underlying base of subgrade. Several factors have been noted to affect the development of rutting. Another type of distresses is cracking. Design of dense graded asphalt concrete mixtures must be balanced between rutting and cracking. Some preliminary findings are shown in the following:

(1) One factor affecting rutting in AC pavement is tire pressure. As the tire pressure increases the potential for rutting also increases. The future performance of new mix designs to account for the increased tire pressure will be unpredictable unless adequate testing and theoretical models are developed.

(2) Another factor for rutting is degree of consolidation. Poor compaction may lead to rutting because of kneading action of traffic loads concentrated in the wheel paths.

(3) Asphalt cement may flow under repetitive loads. Excess asphalt in the mix leads permanent deformation while insufficient asphalt leads to cracking or raveling.

(4) Plastic flow is affected by the size, shape, and texture of the aggregates. High concentration of natural (uncrushed) sand can be large contributor to rut development. The Federal Highway Administration (FHWA) recommends that natural sand content be limited to 15 to 20 percent of the total weight of the mix to minimize the rutting potential on high-volume roadway. However, adequate test methods need to be developed to objectively evaluate the texture and elongation of aggregates.

5.2 TESTS PROPOSED TO EVALUATE THE EFFECTS OF COARSE AGGREGATES ON RUTTING

A theoretical approach was developed to evaluate the rut-resistance. This approach combines a model describing the creep and recovery behavior of AC mixtures with a permanent strain response model such that material parameters obtained from a single creep and recovery test of an AC specimen can be used to predict permanent deformation of the specimen under repetitive loads. Also, the surface deflection of the pavement under load can be calculated with the material parameters obtained. It has been shown in a previous research project that effects of sand on rut-resistance can be evaluated by a parameter - "p" value in the developed model. Similar tests for measuring the effects of coarse aggregates on rut-resistance are being arranged under this project.