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16. Abstract

There has been concern that the legislative mandate to use waste rubber in paving applications will result in a severe environmental problem when it becomes necessary to recycle these pavements. If successful recycling is possible, the long term performance of these pavements becomes a concern. The results of this study indicate that it is possible to recycle this material. However, some techniques for conventional asphalt mixture design, material processing, and construction must be modified to ensure this success, and some techniques may not be appropriate when waste rubber is present in the mixture to be recycled. Many of the results presented in this study are based on experiences in Tyler and San Antonio, Texas, where two of the earliest crumb rubber recycling operations in the United States have transpired.

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by

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IMPLEMENTATION STATEMENT

Study results provide the department with evidence that plant recycling of crumb rubber modified asphalt pavements is possible with RAP contents up to 30 percent. These results are based on a recycling project in the San Antonio District on Interstate 10. The recycle was required because of a premature failure of a crumb rubber modified overlay. The continued good performance of the materials placed in the last portion of the original overlay indicates that the design philosophy used for the recycle operation could improve virgin CRM material as well. These sections were designed using a coarse matrix, high binder concept, and they are still performing well.

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the opinions, findings, and conclusions presented herein. The contents do not necessarily reflect the official views or policies of the Texas Department of Transportation or the Federal Highway Administration (FHWA). This report does not constitute a standard, specification, or regulation, nor is it intended for construction, bidding, or permit purposes.

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The engineer in charge of the study was William W. Crockford, PhD, P.E. #67547.

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ABBREVIATIONS

AC	Asphalt Content
AASHTO	American Association of State Highway and Transportation Officials
CMHB	Coarse Matrix High Binder
CRM	Crumb Rubber Modifier
DOT	Department of Transportation
FWD	Falling Weight Deflectometer
FHWA	Federal Highway Administration
ISTEA	Intermodal Surface Transportation Efficiency Act
POV	Pressure Oxygen Vessel
SH	State Highway
SMA	Stone Matrix Asphalt
TxDOT	Texas Department of Transportation
VPD	Vehicles Per Day

SUMMARY

At the start of this study, few states, with notable exceptions such as Arizona and Florida (and Ontario, Canada), had much experience with using waste tire rubber, and all states were facing a legislative mandate to use specific minimum quantities of this material in paving applications. Justifiable concern was expressed concerning long-term performance of these pavements and the recyclability of failed pavements made with an unfamiliar and essentially unproven material and process. The results of this study suggest that the material is recyclable and that the recycled material, if properly designed and constructed, should have acceptable long-term performance. It also suggests that, at least in Texas, proper design and construction requires some adjustment of normal procedures used for conventional asphalt concrete. For instance, conventional hot in-place recycling equipment without emission control systems are not acceptable for this material. Mix design procedures must take the rubber into account both in the design of the blended aggregate gradation *and* in the design of the blended binder. This report presents a proposed mix design procedure. It recommends existing TxDOT tests methods and specifications for use where possible. The proposed mix design procedure is suggested for use in conjuction with these existing specifications and procedures.

It appears that leaching of harmful materials into groundwater systems is no worse with rubber modified asphalt than with conventional asphalt and that current drinking water standards are maintained in leachate tests. Air quality does not seem to be any more severe a problem than it is with conventional asphalt. However, a common difficulty with air emission studies appears to be that experiment design and adhering to the design in field experiments leaves something to be desired. In some cases, the rubber actually seems to reduce certain emissions, while in others it may increase emissions over conventional asphalt. Statistically, CRM RAP seems no worse than conventional asphalt in the hot mix plant, but there also appears to be potential for confounding in the analysis.

Permeability measurements in the laboratory and radar measurements in the field indicate that the CRM RAP material is not impermeable. Since this is a characteristic of virtually all asphalt concrete materials, blame should not be placed on the rubber or the CRM RAP in particular. This is merely another demonstration of concepts that have led important leaders such as H.R. Cedergren to champion the incorporation of drainage evaluation in the design phase. An inlay type of design appears to be inappropriate in this application in which no positive drainage system has been incorporated.

I. INTRODUCTION

Interest in developing alternative uses for scrap tires has emerged from the enormous quantities of waste tires currently stockpiled in the U.S. An estimated 285 million waste car and truck tires are discarded annually. Of that figure, 33 million are retreaded, 22 million are resold, and 42 million are incorporated in alternative uses. The remaining 188 million tires are added to stockpiles, landfills, or illegal dumps and are considered scrap (Hughes 1993). This annual dumping has resulted in an estimated 2 to 3 billion tires that are currently in stockpiles, which pose two significant threats to the public: as a fire hazard because blazing stockpiles are difficult to extinguish, and as a health hazard when water ponds in tires and provides an ideal breeding ground for mosquitoes.

To date, researchers have experimented with a wide range of technologies for disposing of large quantities of scrap tires. One important application is the combustion of tires for fuel energy in industries such as cement kilns and paper mills, which in 1992 consumed 57 million scrap tires (Hughes 1993). Another technology recycles ground tire rubber in the processing of other rubber products, for which a relatively small market exists. These efforts are, for the most part, the initiative of manufacturing and processing industries.

The growing dilemma of waste tire management has impacted not only the manufacturers and processors, but service agencies, which are major consumers of raw and processed materials as well. In highway construction, the development of technology to use rubber tires is a priority. In whole tire applications, scrap tires are valuable as backfill material, retaining walls, drainage layers, subgrade insulation, and subgrade support. Also, ground tire rubber, which in this document will be termed crumb rubber modifier, can be utilized as an additive or aggregate in the mix design of asphalt pavement. A growing number of state transportation departments use this application. While legislation in at least 44 states has mandated the recycling of tire rubber, it is the federal government that has enacted legislation of the greatest impact to scrap tire use in hot mix asphalt (FHWA 1993). The Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA) specifically addresses the use of crumb rubber modifier (CRM) by requiring the Department of

Transportation (DOT) and the Environmental Protection Agency (EPA) to study and determine:

1. The performance, recycling, and health and environmental aspects of CRM; and
2. A minimum CRM utilization requirement beginning in 1994.

The implementation schedule has been pushed back by recent legislation.

Asphalt treated with CRM is technically described as "crumb rubber modified asphalt," but for the sake of simplicity the term "asphalt rubber" will be used here. In validating the use of asphalt rubber, researchers should anticipate that this pavement material will eventually require recycling either at the end of its life span or at some intermediate stage when rehabilitation or reconstruction becomes necessary. It is thus vital to determine not only the suitability of the asphalt rubber as a virgin material, but also to evaluate the performance of this material when used in its second generation, as a recycled material. The performance of reclaimed asphalt rubber pavement (RAP) and the environmental impact thereof is the focus of this study.

The principal objectives of this study were to:

- Identify potential problems with current mix design and construction techniques that might preclude the possibility of successfully recycling pavements containing rubber.
- Develop recommendations to resolve any problems identified.
- Develop recycling guidelines for department use.
- Evaluate alternative uses for rubber in transportation applications.

The project was undertaken by the Texas Transportation Institute (TTI) in conjunction with the Texas Department of Transportation (TxDOT). This study has monitored CRM mixtures paved in 1992 and 1993 in San Antonio, Texas. The premature failure of the pavement within one year was followed by a recycling phase, during which the pavement was milled to provide RAP to be combined with virgin materials for the laydown of a recycle test section on Loop 1604, just northwest of metropolitan San Antonio.

In the construction of CRM pavements, air quality and long-term pavement performance are two major concerns with a significant relationship because air quality is controlled by temperature, while temperature during construction is a key determinant of performance. Because lower temperatures improve air quality, but typically result in poor compaction by standard procedures, the mix design is problematic. Gyratory compaction tests were used to quantify changes in physical characteristics of CRM mixtures during the densification process.

LITERATURE REVIEW

While the pavement industry is always developing improved, alternative materials, research on the use of CRM in asphalt has gained significance only in the last few decades with the birth of a global effort to conserve and recycle resources in all industries. Because most rubberized pavements are not old enough to have provided material for recycling operations, there is essentially no literature on the performance of recycled rubber asphalt. Nonetheless, this report provides a brief survey on conventional recycled asphalt pavement (RAP), as was found relevant to this project.

In 1977, Piggot wrote on the extent of the rubber-asphalt concrete interaction, the effect of rubber on void content, stability, and the flexural properties of asphalt concrete (AC). In this paper, the use of devulcanized rubber is dated back to 1959, where some city streets in London, Ontario were laid with hot mix asphalt (HMA) containing 0.25% rubber. Devulcanizing adds to the cost of cryogenically ground rubber, which at that time cost 7 cents per 0.45 kg (1 lb). Part of this cost increase may be attributed to the author's suggestion that vulcanized rubber needs to interact with asphalt for 30 minutes at 200°C for full effect.

In the rubber-AC mixture, the interaction between organic fluid (the AC) and polymer (the rubber) was thought to be manifested by the swelling of the polymer. To verify this, the viscosity was measured, since any swelling of the rubber should increase the viscosity of the mixture. At low mixing temperatures (110°C - 120°C), 30% vulcanized rubber increased the viscosity by a factor of more than 20, and at 200°C, the effect of prolonged heating on viscosity was minimal. From this, he determined that any swelling of rubber was minimal upon heating, suggesting that there is no significant interaction between rubber and asphalt concrete. Piggot (1977) also found that for mixtures containing vulcanized rubber, Marshall stabilities of rubber mixtures were low, although road tests were good. In addition, flexural tests showed improved ductility.

In 1978, Westerman performed a cost-benefit analysis of tire rubber used in asphalt using the program TIREC, which provided results supporting the following:

1. Alternatives that reduce tire solid waste are economically preferable to engineering resource recovery alternatives;
2. Repair of roads using tire asphalt rubber is the economically preferable large scale end use (disposal method); and
3. Tire resource recovery by pyrolysis (incineration with energy recovery) cannot be operated at a profit.

Due to the high cost of processing, the conclusion from this analysis was that federal solid waste management programs are the best alternative.

This study goes on to suggest the use of worn tire rubber by cryogenic recovery and mixing crumb rubber modifier (CRM) with asphalt in proportions of 25% rubber to 75% asphalt. Asphalt rubber repair projects have been carried out since the late 1960s in Arizona and are now present in almost every state. In 1978, the EPA established a four-year project on asphalt rubber, from which the following conclusion was drawn: asphalt rubber is not economical. In 1978 each tire processed resulted in a $1.19 loss. However, the net benefit was greater than with pyrolysis, incineration and landfill disposal (Westerman 1978).

In 1979 in Texas, FHWA demonstration projects were built in the Waco district and in El Paso, using an experimental seal coat called Overflex, which contained 25% reclaimed tire rubber. The rubber was ground into particles passing the 1.18 mm sieve and retained on the 0.711 mm sieve. After one to two years of service, the pavements were in satisfactory condition, with minimal reflective cracking. Some stripping due to water entrapment was observed, as well as some flushing of the outside lane due to abrasion of the cover stone, requiring resurfacing after two years. The author suggested a longer study time for a complete evaluation of the durability of AC mixtures containing rubber (Hankins 1979).

A study by Estakhri et al. (1992) documented the use of CRM by the Texas Department of Transportation (TxDOT). In order of CRM consumption, the four main applications were: chip seal or stress-absorbing membrane (SAM) construction, stress-absorbing membrane interlayer (SAMI) construction, crack or joint sealing, and hot mixed asphalt concrete (HMAC) pavement construction.

Part of the Texas Senate Bill 1516 of 1989 mandated that if TxDOT used asphalt rubber, it should use scrap tires converted to CRM within the state, and that the department should give preference to bids using CRM if the materials cost did not exceed that of conventional materials by more than 15%. Experimental CRM asphalt pavements were built in McAllen and Amarillo at a cost of $80 and $52 per 907 kg (1 ton), respectively, compared to $35 per 907 kg for conventional mixes. It was concluded that the greatest hesitation in CRM use was the cost. In Florida, cost estimates by the Florida Department of Transportation indicated an increase in cost of $4.80 per 907 kg of mix, about a 15% increase, when using CRM compared to $32 per 907 kg of conventional mix (Page et al. 1992).

In Australia, the field performance of pavements using CRM was evaluated by monitoring test sections laid in 1989. The asphalt mixture used contained 2.5% scrap rubber, and this was compared with control sections that contained no rubber. The evaluation showed that the mix with rubber had a greater resistance to fatigue cracking than conventional mixes (Williamson 1990).

In 1991, Takallou and Takallou wrote about the benefits of recycling waste tire rubber in asphalt pavements. Their report explained that rubber modified mixes to date had not achieved widespread use due to two main factors:

1. Capital cost for modified mixes is higher than for conventional mixes by 40 - 80%; and
2. Highway constructors lack information regarding the properties and performance of rubber-modified mixes.

The report discussed two major techniques used to process CRM in asphalt which are outlined here. Crumb rubber is obtained from one of two sources: from tire buffings or peelings, or from whole-tire processing. When a used tire is buffed of remaining tread, the removed material is sent to processors and is ground to various mesh sizes. Because the material is from the tread portion only, it is free of steel and fabric. The second technique uses CRM from whole-tire processing, in which mechanical granulation equipment grinds the whole tire into rubber particles passing sieve sizes from 6.3 mm to the 0.425 - 0.18 mm mesh. Steel is removed by magnetic separation and free fabric is removed by air vacuum.

Once granulated, CRM is incorporated into asphalt through what have been termed the "Dry Process" and the "Wet Process." The rubber used in the wet process is usually a fine material with 100% passing the 2 mm (No. 10), or even smaller sieve. In the projects discussed here, the wet process was used. The introduction of the rubber has a tendency to reduce the temperature susceptibility of the asphalt binder. While some maintain that the lighter fractions of the asphalt cement are taken up by the rubber and the rubber then swells, others suspect that this swelling occurs only the first time the binder is mixed, and that the swelling process becomes less obvious during recycling of CRM pavements.

Dry Process

In the dry process, which was developed in Sweden in the 1960s, fine rubber particles replace part of the dry aggregate. The need to increase flexibility and durability and to overcome reflective cracks in resurfaced asphalt pavements led to the adaptation in the U.S. of this technology, which is patented under the name Plus Ride. Plus Ride typically uses 3% CRM by weight of the total mix, and an aggregate gradation that has a gap to provide space for rubber granules to be uniformly dispersed throughout the paving mixture. This paving mix usually requires 1.5% - 2.0% more asphalt than conventional mix.

Reports from the Alaska Department of Transportation and Public Facilities showed that Plus Ride gave a good surface texture due to protruding rubber granulate, which gave improved skid resistance over conventional asphalt concrete during icy conditions.

H.B. Takallou (Takallou and Takallou 1991) further advanced the dry process in 1989 by developing the Generic System. In this process, the aggregate gradation is compatible with conventional dense graded aggregate gradations at 1, 2, or 3% rubber, or 10, 20, and 30 kg/metric ton (20, 40 and 60 lb/ton) of mixture. The average net yield of rubber from a used tire, after steel and fabric removal, is 5.4 kg (12 lb). Projects constructed in 1989 in New York and Canada used 30 kg/metric ton (60 lb/ton), requiring the recycling of five used tires per ton of mixture.

Modifications for Mix Production and Laydown

While batch, continuous, and drum-dryer plants have been used, the batch plant, where quantities of rubber, asphalt and aggregate can be measured exactly and added separately to the pugmill mixing chamber, is preferred. In continuous and drum-dryer mixing, the operation goes on continuously rather than in batches, and the rubber is added from a separate bin with a belt feed. Any process operations require no modification or addition to the conventional equipment. Laydown and compaction equipment for CRM asphalt is the same as conventional equipment.

Wet Process

In this alternative, hot asphalt cement is mixed with a known percentage of CRM by weight of the binder. Experimental work and field trials in several states (Arizona, California, and Colorado) have used 18 - 22% finely ground CRM, passing the 1.18 mm (No. 16) sieve and retained on the 0.6 mm (No. 30) sieve, reacted with various grades of asphalt. At 149°C (300°F) - 204°C (400°F) for periods of 30 minutes to 1 hour, the reaction forms a thick elastic-type material which is diluted with 5% kerosene to aid in application. The result is a thick slurry, which at room temperature becomes a tough rubbery, and elastic binder material. Those rubber particles which are undissolved behave as aggregate, and the dissolved ones modify the binder.

The production of this mix differs from that of conventional mix in that the rubber is pre-blended with the asphalt in insulated trucks and tanks. Also, the production temperatures for CRM mixes are higher than for typical mixes, i.e., 177°C (350°F) - 204°C (400°F). The asphalt and CRM are combined and mixed in the blender unit, pumped into an agitated storage tank, then reacted for a minimum of 45 minutes from the time the rubber is added. The wet process requires specialized equipment such as a heating tank, an asphalt rubber blender for homogenous mixing, and an asphalt rubber storage tank which maintains the right temperature.

An economic analysis showed that high initial and capital costs, a lack of information transfer between states, and the lack of used-tire processing technology have hindered widespread use of rubberized asphalt in the U.S. The leading conclusion in this paper was that the generic system proves to be the most promising alternative for CRM asphalt (Takallou 1991).

Recent papers from the Transportation Research Board have investigated the interactions and properties of asphalt rubber mixtures. According to Stroup-Gardiner et al. (1993), the primary goal of the dry process is to provide solid elastomeric inclusions within the asphalt-aggregate matrix. This asphalt rubber interaction is influenced by the concentration of rubber present, binder grade and binder chemistry, type of rubber (natural vs. synthetic), viscosity as affected by aging, and pretreatment of the rubber.

In a study by Khedaywi et al. (1993) to determine asphalt rubber properties, mixtures were prepared using 0, 5, 10, 15, and 20% rubber by weight of the mix. In evaluating the effect of rubber content on physical properties, it was found that Marshall stability decreased with increasing rubber content, while flow and voids in the mineral aggregate (VMA) increased.

Recent findings from crumb rubber use have been both promising and discouraging. In a report to Congress in 1994, the EPA and FHWA concluded that:

1) There is no reliable evidence that the manufacture, application or use of CRM asphalt increases the threat to human health or to the environment, compared to conventional mixes;
2) No evidence shows that CRM asphalt cannot be recycled as much as conventional HMA; and
3) No evidence shows that CRM asphalt does not perform adequately.

Although Section 1038 of ISTEA mandated a minimum use of CRM for 1994, lobbying by state highway officials, aggregate producers, and asphalt contractors has resulted in Congress withholding the necessary funds to implement the legislation (Drake 1994). Among those opposing Section 1038 is the American Association of State Highway and Transportation Officials (AASHTO). AASHTO advocates flexibility in how states choose to dispose of their waste materials, and it finds that the added costs of CRM use exceed the benefits of using the material.

Accordingly, AASHTO has adopted another CRM mandate resolution that requests that Congress allow credit for other scrap tire uses; convert the mandate to an incentive program instead of a sanctioned program; and allow usage waivers to states where the cost of using CRM is greatest. While environmentalists are trying to restore funding, forty-three state highway departments are trying to get Section 1038 repealed.

Initial costs for CRM are 20% - 100% higher than for conventional mix; these costs are expected to decrease if CRM asphalt is used more widely. As for performance, CRM technology (design and construction) is not always correctly applied, which may explain why performance fluctuates from project to project.

Recyclability of CRM Asphalt

CRM RAP is documented in only two projects in North America, and although these pavements have not been in service long enough to evaluate long-term monitoring, performance so far is satisfactory (Drake 1994). Because there is little or no documentation on the recycling of rubberized asphalt pavements, this section discusses literature on conventional reclaimed asphalt. It has been estimated that the demand for RAP will grow by 4.1% per year between 1993 and 1998, as a result of increased waste disposal costs, growing efforts to reclaim solid waste, product innovation, and legislative mandates (Drake 1994). Kari (1979) stressed that hot mix recycling must satisfy the following economic, technical, and environmental needs of the engineer:

1) The operation must utilize existing hot asphalt plant equipment with minimal modifications;
2) Productivity levels thereof should be at least equal to those from conventional mixes;
3) Stability of the mix should be comparable to conventional mix stabilities; and
4) The operation should be environmentally acceptable.

Cold milling is another important recycling technique, defined as the process of partial depth removal and profiling of asphaltic and/or portland cement concrete pavement by grinding or milling (Van Deusen 1979). No effective equipment existed for this technology until 1976. Assuming that the original mix had a quality aggregate (i.e., not so soft as to result in too many fines during milling), the material resulting from the cold milling operation of an asphalt concrete surface is usually of equal or even superior quality, because of a higher percentage of crushed material in the milled aggregate. Also, this aggregate is partially coated with asphalt cement, thus ensuring thick films of binder and thus greater durability.

A more recent paper (Better Roads 1993) deals with the current status of RAP recycling, describing the four main recycling methods: cold planing, hot recycling, cold in-place recycling, and full-depth reclamation. **Cold Planing** is automatically-controlled removal of asphalt pavement to a desired depth and the restoration of the surface to a desired grade and slope, or to a desired surface texture to improve skid resistance. A self-propelled rotary drum cold planing machine is used, and the RAP is transferred to trucks for stockpiling. In **Hot Recycling,** RAP from cold planing or a crushing operation is combined with new aggregate and asphalt concrete or recycling agent to produce hot mix asphalt. Both batch and drum-type plants can be used, and the placement and compaction of the product are the same as for conventional HMA. The ratio of RAP to virgin aggregate depends on the final mix properties desired and the type of hot mix plant. Typical blends of RAP to virgin materials are 10:90 and 30:70, with a maximum of 50:50. Thanks to recent microwave technology (e.g., Cyclean) for heating and modifications to conventional processes (e.g., Rapmaster), the use of up to 100% RAP without smoke problems is becoming a reality. **Hot In-place Recyling** involves heating and scarifying deteriorated asphalt pavement to a specified depth, and then adding new hot mix and a recycling agent to the RAP. **Cold In-place Recycling** involves pulverizing existing pavement without heat, and in **Full Depth Reclamation**, all the asphalt pavement and some of the underlying material are treated to produce a stabilized base course.

A survey (Estakhri et al. 1992) of routine RAP use primarily in Texas and a laboratory study determined the following:

1) Conventional mix design does not always apply to mixes of 100% RAP, perhaps because the RAP is already at the optimum AC content;
2) Properties of RAP are significantly improved when blended with virgin asphalt mixtures;
3) RAP mixtures are generally more susceptible to moisture damage than virgin asphalt mixtures; and
4) Hveem stability appeared to be the best test for characterizing RAP and RAP blends in terms of expected performance.

LEGISLATIVE MANDATE FOR USE OF WASTE TIRE RUBBER

The passage of the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991, Section 1038, set the stage for state mandates on the use of CRM in asphalt pavements. The provisions of this legislation required the DOT and the EPA to conduct a study to determine (1) the threat to human health and the environment associated with the production and use of asphalt pavement containing recycled rubber, (2) the degree to which asphalt pavement containing recycled rubber can be recycled, and (3) the performance of the asphalt pavement containing recycled rubber under various climate and use conditions. The term "asphalt pavement containing recycled rubber" means any hot mix or spray applied binder in asphalt paving mixture that contains rubber from whole scrap tires which is used for asphalt pavement base, surface course or interlayer, or other road uses, and contains not less than 9 kg (20 lbs) of recycled rubber per 907 kg (1 ton) of hot mix or 136 kg (300 lb) of recycled rubber per 907 kg (1 ton) of spray applied binder.

In addition, the DOT and state departments would jointly conduct a study to determine the economic savings, technical performance qualities, threats to human health and the environment, and environmental benefits of using recycled materials in highway devices and appurtenances and highway projects. This would include an examination of how states use various technologies and the current practices of all states relating to the reuse and disposal of materials in federally assisted highway projects.

The legislation Section 1038 requires each state to use a minimum percentage of recycled rubber in each ton of asphalt pavement laid in the state and financed in whole or in part by any assistance pursuant to title 23, United States Code. Beginning on January 1, 1995 the requirement shall be: (a) 5 percent for the year 1994, (b) 10 percent for the year 1995, (c) 15 percent for the year 1996, and (d) 20 percent for the year 1997 and each year thereafter.

The Secretary of the DOT may waive the utilization requirement for any 3-year period on a determination that there is reliable evidence that (a) the manufacture, application, or use of asphalt containing recycled rubber substantially increases the threat to human health or the environment as compared to the threats associated with conventional pavement, (b) asphalt pavement containing recycled rubber cannot be recycled to substantially the same degree as conventional pavement, or (c) asphalt pavement containing recycled rubber does not perform adequately as a material for the construction or surfacing of highways and roads.

One of the main purposes of incorporating ground tire rubber into asphalt concrete, at least from a political and environmental standpoint, is to reduce the solid waste problem. One might expect that the cost of such usage of waste materials would be quite reasonable.

Such is not the case. The costs are quite inflated for new CRM mixtures. The original San Antonio job had two bid items, one CRM mix (18% crumb rubber by weight of the binder) and one standard mix without rubber. The cost of the conventional mix (1992) was $23 per 907 kg for 7,711,070 kg (8,500 tons), while the cost of the CRM mix was $45 per 907 kg for 23,586,803 kg (26,000 tons). The cost of the recycle with 30% RAP (1993) was $31.50 per 907 kg for 17,236,510 kg (19,000 tons).

SURVEY OF STATES' EXPERIENCE WITH CRM PAVEMENTS

Highway departments in the US and Canada were surveyed to determine if any state had experience with recycling asphalt rubber mixtures. While most states reported in 1992 that they had built at least one CRM pavement, plans to recycle were only documented in Arkansas, California, Illinois, and New Hampshire. Results from the survey are shown in Table 1a.

The study supervisor made several contacts with Canadian researchers from the Ontario Ministry of Transport and the National Research Council in Ottawa. A study completed in Canada that was similar to our project examined compaction problems with CRM use and provided an environmental assessment thereof.

A survey was conducted for all districts in Texas, from which certain districts were identified as having candidate sites to conduct recycling. The study supervisor met with District 10 personnel and obtained milled material from an asphalt rubber pavement on I-20 between Longview and Tyler. In a limited evaluation of the milled material and the layer below, the rubber mix appeared to be slightly stripped. Table 1b shows the experience of Texas districts with CRM use, as documented in the summer of 1993. Most districts have existing CRM pavements and plan to recycle them in the future. Those districts that do not already have CRM pavements generally have plans to experiment with them in the future. The existing pavements have used CRM either in hot mix, seal coat, or crack seal applications. District 15 (San Antonio) reported extreme difficulty faced in using CRM asphalt in conventional mix design. On a lighter note, the same district reported experience with hot in-place recycling of a CRM asphalt layer to be 90% successful.

TTI was present at a CRM recycling job on loop 1604 and on IH-10 in San Antonio during 1993. No notification was received for the hot in-place recycle in Tyler, so TTI was not present for that operation. However, pictures presented by the Materials and Tests Division indicated, at least subjectively, that there was a significant problem with air emissions (opacity, i.e., smoke). Standard hot in-place recycling equipment is not recommended for this material. However, equipment such as Pyrotech with emission controls on the equipment train might be successful in hot in-place recycling operations. The counter flow drum plant used in San Antonio seemed to work quite well in all facets of that recycling operation. It appears that there have been so many construction problems associated with each CRM job that it is difficult to say whether performance problems should be attributed to mix design or to construction practice.

Table 1a. Survey of CRM Use in State Highway Departments.

State	#CRM Roads Built	Test Section	Specs For CR, RB, RM	Mix Design	Recycled Roads	Success Rate	Test Section?	Plans to Recycle	When Scheduled	Location	Test Section?	RAP Owner-ship
Alabama	0	N	N	N	N			N				C
Alaska	Many	S	Y	S	N			N				S
Arizona	Many	S	Y	M	N			N				E
Arkansas	1	N	N	N	Y	<20%		Y	1 yr	Russell-wood	Y	E
California	Many	S	Y	S	N			Y	1 yr	I40, L.A.	Y	C
Colorado	0	N	N	N	N			N				C
Connecticut	Few	N	N	N	N			N				E
Delaware	0	N	N	N	N			N				C
Florida	4	Y	Y	Y	Y	Good	Y	N				C
Georgia	2	Y	S	M	N			N				C
Hawaii	0	N	N	N	N			N				C
Idaho	0	N	S	H	N			Y	10 yrs	Cent.Ill.	Y	S
Illinois	2	Y	S	S	N			N				C
Indiana	1	N	S	H	N			N				C
Iowa	6	Y	Y	M	Y	Moderate		N				C
Kansas	6	Y	Y	S	Y	40-60%		N				E
Kentucky	0	N	N	N	N			N				C
Louisiana	2	S	S	S	N			Y	5 yrs	Shreve P.	N	S
Maine	1	Y	S	H	N			N				C
Maryland	1	Y	S	S	N			N				C
Massachusetts	0	N	N	N	N			N				E
Michigan	3	S	S	S	Y	20-40%		N				C
Minnesota	0	N	N	N	N			N				C
Mississippi	Few	N	Y	N	N			N				C
Missouri	0	N	N	N	N			N				C
Montana	1	S	S	H	N			N				E
Nebraska	0	N	N	N	N			N				S
Nevada	1	N	N	N	N			Y	10-15 yrs			C
New Hampshire	2	N	S	M	N			Y	5 yrs			S
New Jersey	1	N	S	M	N			N				C
New Mexico	0	N	N	N	N			N				S
New York	Many	Y	Y	Y	Y	Moderate	S	Y	5-10 yrs	All Over	S	C
North Carolina	2	S	S	N	N			N				C
North Dakota	Few	N	S	N	N			N				E
Ohio	4	S	Y	Y	Y	60%		N				C
Oklahoma	1	Y	S	N	N			N				C
Oregon	0	N	N	N	N			N				S
Pennsylvania	Many	S	Y	Y	Y	Good	S	Y	3 Yrs	Scranton	S	C
Rhode Island	0	N	N	N	N			N				S

Table 1a. (Cont.)

State	#CRM Roads Built	Test Section	Specs For CR, RB, RM	Mix Design	Recycled Roads	Success Rate	Test Section?	Plans to Recycle	When Scheduled	Location	Test Section?	RAP Owner-ship
South Carolina	1	Y	Y	N	N			N				C
South Dakota	Few	Y	S	N	N			N				S
Tennessee	0	N	N	N	N			N				C
Utah	Few	N	N	N	N			N				S
Vermont	0	N	N	N	N			N				S
Virginia	1	N	S	N	N			N				C
Washington	5	Y	Y	N	N			N				E
West Virginia	0	N	N	N	N							S
Wisconsin	0	N	N	N	N							C
Wyoming	4	N	N	N	N			N				C

Y= Yes; N= No; S= Some; E= Either; Su= Supplemental; St= STATE; C= Contractor
M= Marshall; H= Hveem

Table 1b. Survey of CRM Use in Texas Districts.

District	Existing CRM Pavements	Type (HM, SC, Other)	Future Plans to Use CRM (New or Recycled)?	Ever Recycled CRM Pavements	Success Rate (%)	Max % RAP Used
Abilene (8)	Y	SC	Aug 1993	-		0
Amarillo (4)	Y	HM, SC	-	N		-
Atlanta (19)	Y	SC	Aug 1994	N		
Austin (14)	Y	SC	N	N		0
Beaumont (20)	Y	SC	1994	N		75
Brownwood (23) *						
Bryan (17) *						
Childress (25)	N	-	Future	N		20
Corpus Christi (16)	Y	SC	Sep 1993	N		0
Dallas (18) *						
El Paso (24)	Y	SC	Future	N		0
Fort Worth (2)	N	-	Never			35
Houston (12)	Y	SC	1996	N		100
Laredo (22) *						
Lubbock (5)	Y	HM	Future	N		30
Lufkin (11) *						
Odessa (6)	Y	SC	1994	N		0
Paris (1)	Y	-	Future	N		0
Pharr (21)	Y	SC	-	N		50
San Angelo (7)	N	-	Future	N		0
San Antonio (15)	Y	HM, SC	-	Y	90	60
Tyler (10)	Y	HM, SC	Mar 1993	Y	25	0
Waco (9)	N	-	-	N		25
Wichita Falls (3)	Y	HM, SC	N	N		0
Yoakum (13)	Y	SC	Future	N		100

- no info. provided
* survey not submitted

Y=Yes; N=No
HM=Hot Mix; SC=Seal Coat

OVERVIEW OF SEQUENCE OF CONSTRUCTION

To address construction and performance concerns in this project, experimental CRM asphalt pavements were placed northwest of San Antonio. Figures 1-3 show the traffic situation in this area. It is significant that the peak traffic counts occur during the hot part of the day as well as during the hot part of the year. This indicates a need to reduce temperature susceptibility of the binder as much as possible to balance resistance to rutting and cracking.

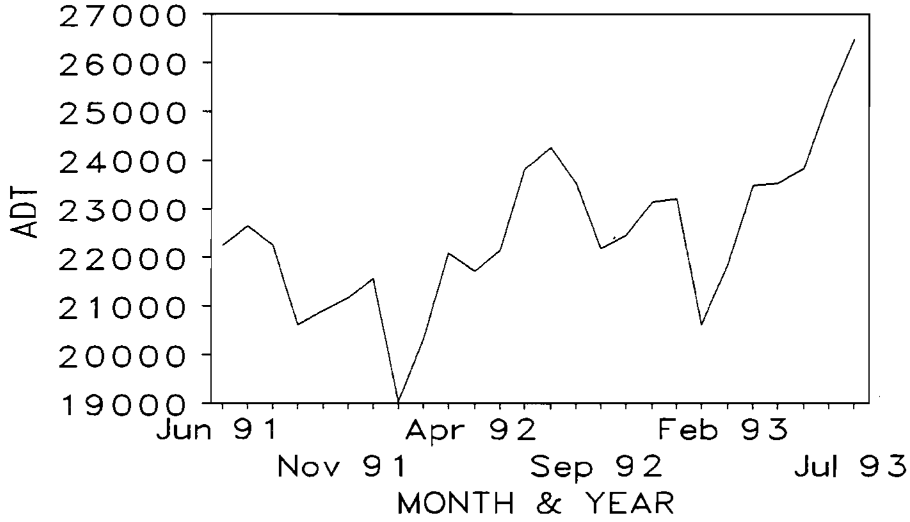

Figure 1. IH-10 Recent Traffic Growth.

The construction activities were implemented to provide data for the following:

- An evaluation of the compaction and densification of asphalt rubber mixtures;
- An evaluation of the environmental effects of recycled asphalt rubber resulting from the hot mix process, and from stockpiling the reclaimed pavement (air quality, surface runoff, and groundwater quality); and
- An assessment of the permeability characteristics of asphalt rubber mixtures.

In July of 1992, a hot mix asphalt concrete overlay containing a wet process binder was placed on IH-10, just northwest of San Antonio. By 1993, the new overlay failed. The

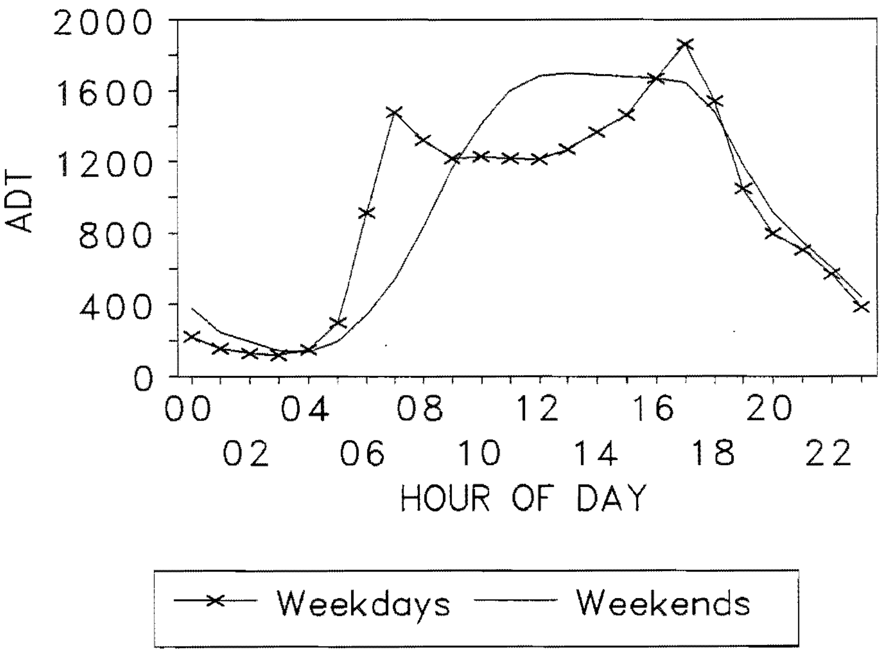

Figure 2. IH-10 Daily Traffic Counts.

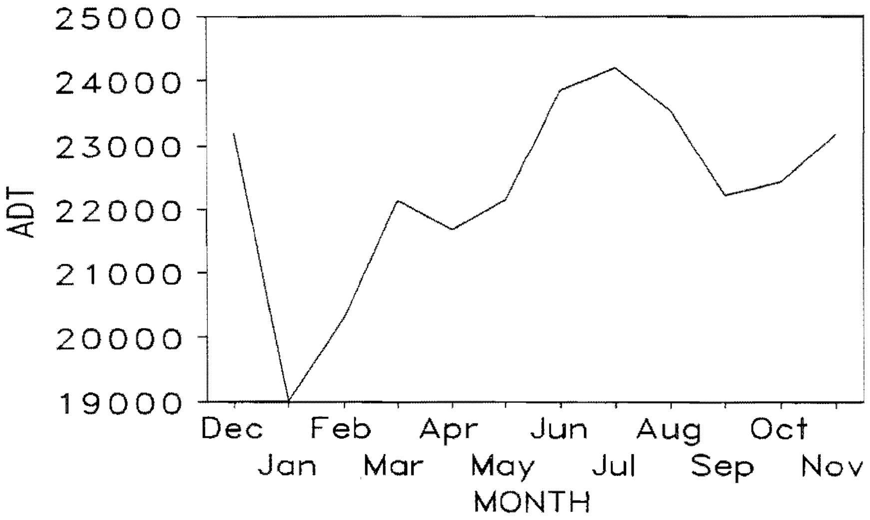

Figure 3. IH-10 Monthly Traffic Counts.

signs of distress included alligator cracking, rutting, and pumping of fine material. Right wheel path rut depths at four locations averaged between 0.8 and 14.7 mm (0.03-0.58 in.) with the deepest rut measurement being 27.7 mm (1.09 in.). These same symptoms were noticed at other locations, including on a section of IH-20 between Longview and Tyler. Some have attributed these problem areas to the rubber in the mixtures. However, the presence of rubber alone cannot be the sole perpetrator of the problems as the following observations show: (1) many overlays that do not contain any rubber at all exhibit the same symptoms of distress which may be related not only to the material characteristics but also to characteristics of the bond between the overlay and the previously existing surface, and (2) rubberized seal coats and some hot mix asphalt concrete (*e.g.*, Loop 323 in Tyler, and the frontage road on IH-10) appear to be performing well.

On IH-20 in Tyler, a small size, dense graded siliceous aggregate was used in the mix. On the original IH-10 job, a small size aggregate was used at the start of the job, but by the end of the job, a larger limestone aggregate was in use and the gradation had been changed from dense to what may be described as a gap graded material, and the binder content was increased to give the desired film thickness. This basic concept for ensuring stone on stone contact throughout the aggregate fraction retained on the 2 mm (No. 10) sieve, and for ensuring sufficient binder to give the desired film thickness to resist environmental damage is the background for the material that the Texas Department of Transportation (TxDOT) now refers to as CMHB (Coarse Matrix, High Binder) mixtures. It turns out that one of the advantages of the rubber in the mix is that it helps to prevent drain-down of the asphalt binder in these high binder mixtures.

By the end of the San Antonio overlay job, it had become apparent that the coarser matrix material would be a better performer. Not only was the stone skeleton more substantial in this mix, but the compaction process was better as well. The finer, dense graded material used at the start of the IH-10 job was difficult to compact and resulted in high and spatially variable air void contents, probably due to both rebound of the rubberized mix and bridging across the uneven transverse profile of the previously existing surface by the drum rollers. It is thought that the high air void contents connote higher permeabilities which can lead to moisture assisted damage in the layer of interest as well as at the interface between the overlay and the previously existing pavement. When the overlay later failed, the starting point for the new mix design was the coarse, gap graded material used at the end of the first job. TxDOT conducted extensive laboratory work on the designs and placed a test section on Loop 1604 in the spring of 1993. Based on the favorable results from the last portion of the first IH-10 job, the Loop 1604 test section, and additional laboratory testing, the decision was made to recycle the mix in the failed sections on IH-10. The plan was to mill the outer lane of the failed material and to take this reclaimed asphalt to a plant where it would be added to new aggregate and asphalt in such a way that a material similar to a CMHB would result for placement in the inlay. The plan was implemented in the fall of 1993, and the material is performing satisfactorily at this time. The surface texture is coarse, and water drains from the pavement for a considerable time after a rainfall event, but no distress is apparent at this time. The final mix used on the recycle contained 30% RAP, 5.7% asphalt, and 79.6% (by weight) aggregate retained on the No. 10 (2 mm) sieve.

II. PROCESS AND MATERIALS EVALUATION

ENVIRONMENTAL ISSUES

There has been some concern that the mandated use of waste tire rubber will create an even worse environmental problem when the material must be recycled in the future than now exists. Specifically, air quality might be adversely affected during the production process, and water quality might be affected due to leaching in RAP stockpiles and under the pavements. Aged or weathered asphalt pavement is different from new asphalt pavement, and the addition of modifying agents to restore asphalt properties will change the chemical composition of manufacturing and application emissions. In general, aging is accompanied by reactions which essentially increase the asphaltene content. Asphaltenes are large, complex nonpolar molecules (Bloomquist 1993).

While the number of detections possible in a volatile emissions sampling operation can be phenomenal, the evaluation of their impact concentrated on a few materials are known to be hazardous to the environment. Among these are polyaromatic hydrocarbons (PAHs) which include naphthalene, fluorene, anthracene and benzopyrenes. Other compounds are volatile organic compounds (VOCs), benzene, styrene, 1-2 butadiene, phenanthrene, and particulates (Bloomquist 1993).

For the first generation of the IH-10 construction, the CRM asphalt concrete was produced by the wet process in a drum mixer at the Redland Stone Products Company in San Antonio. Southwestern Laboratories (SWL) sampled the hot mix operation and tested for air emissions using standard EPA sampling techniques. The plant was equipped with a baghouse rather than a scrubber and operated at a production rate of 351,080 kg/hr (387 tons/hr) during the sampling. For comparison, samples were also taken at the Duininck Brothers hot mix plant. Testing was achieved in three separate sampling trains. Condition 1 was at a high temperature of 163°C (325°F) with a CRM mix; Condition 2 was at a low temperature of 149°C (300°F) with a CRM mix; and Condition 3 was at a high temperature with a conventional mix containing no CRM. Three trials were conducted for each test condition. Trials for volatile organic sampling train (VOST) chemicals lasted 20 minutes, and trials for other compounds lasted 60 minutes. Emission rates were calculated as pounds per hour.

Emission rates at the Redland Stone plant are shown in Appendix B. The only semivolatile organic chemicals detected in the conventional hot mix were 2-methylnaphthalene, naphthalene, and phenanthrene. Emissions of VOST compounds decreased with temperature and were higher during the mixing of CRM asphalt concrete than during the conventional mix operation. A statistical analysis of variance of the air emissions data showed that overall, there is very little difference in emissions from CRM plants versus standard asphalt plants, as shown in Table 2.

Table 2. Statistical Analysis of Air Emissions Data at Two Texas CRM Plants.

Factor	VOC	Benzene	Styrene	Naphthalene	2-Methyl-Naphthalene	Phenanthrene	Butadiene	Particulates	Opacity
Plant	N	S	N	N	S	S	S	N	N
Temperature	N	S	N	N	N	S	N	N	N
%CRM	N	N	N	N	N	S	N	N	N
N = not statistically different, S = statistically different									

The only case in which 18% rubber resulted in higher emissions than no rubber was for phenanthrene, but this may be attributed to the fact that 44% of the total possible number of observations was missing for this compound. In some cases, measurements showed a higher concentration of a compound at the low temperature condition than at the high temperature condition, which is highly unlikely. A discrepancy of this sort may well be the result of some variation in the hot mix plant operation itself or in the experimental technique, rather than the chemistry of the materials. It should be noted that hot-mix asphalt production is, by nature, a highly variable process, dependent on parameters such as the fueling rate of the dryer, mix temperature, asphalt throughput rate, and asphalt binder content, which are all themselves subject to variation (Bloomquist 1993).

In light of this high variability, it can at least be argued that for most chemicals, the effect of CRM on emissions may be relatively small compared to the effects of other variables. In two Ontario studies of environmental emissions, the levels of PAH emissions were higher during the mixing of rubber-modified asphalt concrete than during the mixing of conventional hot mix asphalt, while VOC emissions were lower. A Parmer County, Texas, study showed that VOC emissions were slightly lower for the rubber mixture. The limited sampling performed in this study was inadequate to assess emissions from mixing of asphalt pavements with satisfactory precision, as is shown by the erratic nature of the sampling results. Extensive sampling would be required to determine emission rates with the degree of precision necessary to differentiate between emissions from conventional and modified asphalt pavements.

Southwestern Laboratories (SWL) also performed leachate testing to determine the potential for CRM to contaminate surface runoff and groundwater. From the recycled asphalt rubber stockpiled at the Colorado Materials site in New Braunfels, Texas, a sample was transported to the laboratory and was subjected to a simulated precipitation leachate which is expected to represent the lifetime cumulative effect of acid rainwater leaching. The simulated precipitation leachates were analyzed for trace metals, volatile organic compounds, and semivolatile organic compounds.

Results from the leachate analysis showed that the only constituent occurring at levels above the detection limit was mercury, which was detected at 0.0011 mg/l. This level is below the current EPA drinking water limit for mercury (0.002 mg/l). A table of all the trace metals tested for is provided in Appendix B, which shows that all other compounds tested for were below the analytical detection limit. From these findings, it can be concluded that trace metals, volatile organics, and semivolatile organics may be leached from asphalt rubber, but at levels too low to be environmentally significant or hazardous.

SWL performed emissions sampling for the recycled CRM mix at the Colorado Materials Company site in New Braunfels, Texas, during the months of October and November 1993. Conditions were similar to the Redland Stone operation: Condition 1 - low temperature - <149°C (300°F) with recycled CRM; Condition 2 - high temperature >149°C (300°F) with recycled CRM; Condition 3 - high temperature >149°C (300°F) with conventional hot mix asphalt. The CRM mix consisted of 30% RAP, and both mixes had an average asphalt content of 5.3%. Because temperature requirements at the field site limited the low temperature range of the mix, no testing was performed on Condition 1.

PERFORMANCE RELATED CHARACTERISTICS

Original IH-10

Most of the studies concerning long-term performance of the mix were done in the laboratory. The original IH-10 overlay built in July 1992 had a mixture design consisting of 7.5% AC-10 and 18% crumb rubber by weight of the binder in the wet process. During construction, samples of hot mix CRM asphalt were obtained and sieve analyses were run in the field lab. Cores molded from the samples were also tested for density and specific gravity. The mixture varied from a dense gradation (59% retained on the No. 10 sieve) to a gap gradation (81.1% retained on the No. 10 sieve) at the end of the job, with asphalt contents ranging from 7.0 to 8.5%. Figures 4 and 5 show a comparison of these gradations.

Samples were taken from the truck at Redland Stone to the District Materials and Tests Division Lab, molded into cores, and tested for Hveem stability, Rice specific gravity, and gradations. In San Antonio, residency personnel obtained thirty cores from different positions relative to the wheel path and took nuclear density measurements.

Recycle Test Pavement

David Kight performed experimental mix designs at the Redland Division Laboratory in March of 1993. Two gap graded mixes using 30% RAP were compared. One had 74.3%

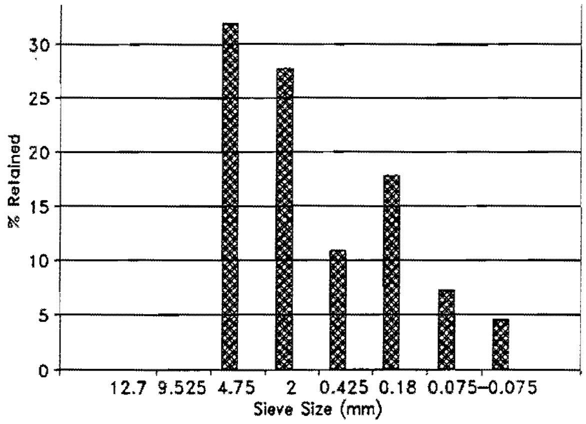

Figure 4. Dense Gradation Used on Original IH-10 (59.6% Retained on the 2 mm Sieve).

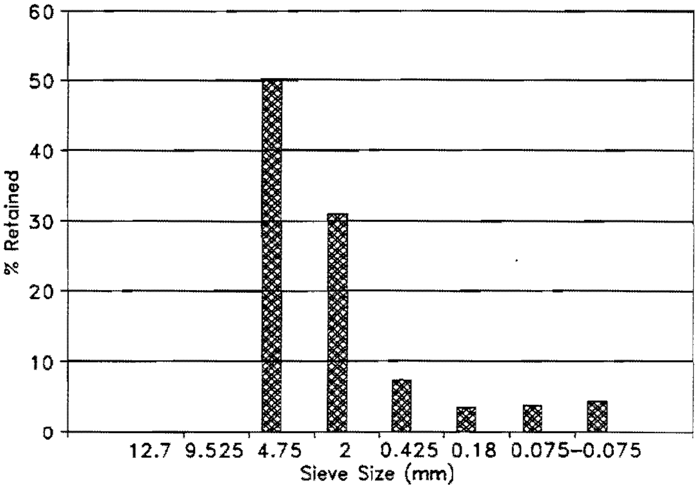

Figure 5. Gap Gradation Used on Original IH-10 (81.1% Retained on the 2 mm Sieve).

+2 mm aggregate while the other had 80.1% +2 mm aggregate, and both had the asphalt content vary at 4.0%, 5.0%, 6.0% and 7.0%. Molded cores were tested for Hveem stability. The results showed that stability increased with an increase in +2 mm material and a decrease in asphalt content. The mix that yielded the highest durability (asphalt content) with an acceptable stability was that having 74.3% +2 mm material at 6.0% asphalt, which gave a stability of 38. Specification values usually range from 30-37 minimum. A test section was constructed on Loop 1604 with a final mix design that contained 30% RAP, 5.8% asphalt cement, and 74.3% limestone aggregate retained on the 2 mm sieve.

CRM Recycle

Prior to the recycling phase, several type "C" CMHB mix designs were experimented with, using 30% RAP, AC-20, and a gap gradation. For a particular mixture containing 79% +2 mm material, five asphalt contents were varied from 4.5% to 6.0%. Hveem stability, Rice specific gravity, and VMA were measured. Finally, in October 1993, the outer lane of the failed overlay was milled and sent to Colorado Materials Co. in New Braunfels, where it was used in a mix containing 30% CRM RAP, 5.7% AC-10, and 79.6% aggregate retained on the 2 mm sieve. Sieve analyses were run on material entering the cold feed bins. Samples from the hot mix operation were molded into cores that were analyzed for gradation and tested for Hveem stability, specific gravity, nuclear asphalt content, and creep. The percent +2 mm aggregate and the Hveem stability were plotted against air voids, creep strain, and creep stiffness, but no definite relationships were found.

Compaction Studies

To quantify changes in physical characteristics of CRM mixtures during the densification process, a factorial design was implemented. Three mix designs and several construction conditions were included in the design, giving a total of 51 tests. The tests performed were in general accordance with the ASTM D3387: Standard Test Method for Compaction and Shear Properties of Bituminous Mixtures by Means of the U.S. Corps of Engineers Gyratory Testing Machine (GTM). The U.S. Army Engineer Waterways Experiment Station (WES) undertook this task.

WES provided data on GTM revolutions, unit weight, and a gyratory stability index (GSI) for each test at a frequency of 10 revolutions for a total test period of 250 revolutions.

Some background on this test procedure is presented here so as to enhance understanding of the significance of the results obtained by WES and the conclusions drawn.

The development of the GTM originated at WES and has continued at the Engineering Developments Company (EDCO) (McRae 1993). The GTM is a combination compaction and plane strain, simple shear testing machine for use on soils, base course materials, and bituminous type-paving materials. The gyratory shear it provides is a more uniform shear strain than the direct shear and triaxial shear achieved with conventional testing equipment. Two of its several applications include:

1) Providing a testing machine in which theoretical vertical stress at any depth within the structure can be introduced for compaction and shear testing; and
2) The production of test specimens by a kneading compaction process which gives stress-strain properties representative of actual compacted bituminous pavement.

TxDOT uses a version of the Hveem stability to evaluate the adequacy of asphalt mixtures. A stability of 35 is often recommended as a minimum for adequate performance. Specification values usually range from 30-37 minimum, depending on the expected traffic for which the pavement is being designed. Because CRM mixtures sometimes have low stabilities when measured with the standard test, and because the stability test does not provide much information with respect to compaction, additional tests were conducted to further explore the material behavior.

Samples of the original IH-10 CRM overlay, the Loop 1604 CMHB and 30% RAP mixes, and the final IH-10 inlay materials were tested in the GTM. For the experiment design, thirteen combinations of three loading pressures and three temperatures were used. The low (L), medium (M), and high (H) pressures were 517 kPa (75 psi), 1034 kPa (150 psi), and 2068 kPa (300 psi), to simulate a range of traffic wheel loads. To simulate truck (T) and roller (R) compaction, loading pressures of 1207 kPa (175 psi) and 1551 kPa (225 psi) were used. The L, M, and H temperatures were 60°C (140°F), 121°C (250°F), and 160°C (320°F) to cover the range of temperatures expected from hot summer pavement temperatures to temperatures during laydown. Table 3 shows the experiment design. Table 4 shows the ending gyratory strain, the GSI, and the GCI for each sample. Figures 6 and 7 illustrate that the mixtures are easy to compact and that there is a relationship, albeit accompanied by high variability, between Hveem stability and the gyratory shear strain (a form of engineering shear strain) at the end of the compaction. The GSI measured in the GTM was greater than 1.0 in all except one test. That test was a 517 kPa (75 psi) test, and our computations indicate that a 36,287 kg (80 kip) truck would be closer to a 1207 kPa (175 psi) vertical pressure in the GTM. Values of GSI greater than 1.0 generally indicate excess asphalt content or some other physical characteristic that results in undesirable plasticity.

Table 3. Factorial Design for Compaction Study.

RAP CONTENT (%)	PRESSURE		TEMPERATURE (°F)		
	(kPa)	(psi)	60°C (140°F)	121°C (250°F)	160°C (320°F)
0	517.1	75		SLM	
	1034	150	SML	SMM	SMH
	2068	300		SHM	
30	517.1	75		SLM	
	1034	150	SML	SMM	SMH
	2068	300		SHM	
100	517.1	75		SLM	
	1034	150		SMM	
	2068	300		SHM	
30	1207	175	STL	STM	
	1551	225	SRL	SRM	

L = low M = medium H= high T = truck R = roller

Table 4. Results of Gyratory Testing.

Specimen ID	CRM RAP Content (%)	Gyratory Angle (degrees)	GCI	GSI	Gg (psi)
SHM0.R1	0	2.81	0.9930	2.23	4247
SHM0.R2	0	2.54	0.9894	2.14	4027
SHM0.R3	0	2.36	0.9894	1.95	3385
SMH0.R1	0	1.82	0.9678	1.51	2403
SMH0.R2	0	1.72	0.9763	1.39	3154
SMH0.R3	0	1.94	0.9763	1.60	3462
SMM0.R1	0	2.13	0.9797	1.68	2915
SMM0.R2	0	2.10	0.9813	1.66	3301
SMM0.R3	0	1.94	0.9784	1.54	3481
SML0.R1	0	1.94	0.9751	1.54	3993
SML0.R2	0	2.01	0.9717	1.51	3933
SML0.R3	0	1.95	0.9721	1.48	3550
SLM0.R1	0	1.34	0.9650	1.01	3759
SLM0.R2	0	1.34	0.9676	1.01	3693
SLM0.R3	0	1.30	0.9650	0.96	4270
SHM3.R1	30	2.61	0.9914	1.79	4126
SHM3.R2	30	2.17	0.9906	1.60	4369
SHM3.R3	30	3.03	0.9916	1.70	6315
SMH3.R1	30	2.26	0.9850	1.54	2376
SMH3.R2	30	2.17	0.9809	1.51	2557
SMH3.R3	30	2.34	0.9863	1.72	1918
SMM3.R1	30	2.26	0.9875	1.61	2563
SMM3.R2	30	2.37	0.9825	1.62	2285
SMM3.R3	30	2.06	0.9771	1.46	3277
SML3.R1	30	2.18	0.9751	1.49	2800
SML3.R2	30	2.44	0.9761	1.64	2672
SML3.R3	30	2.35	0.9775	1.58	2637

Table 4 (cont.)

Specimen ID	CRM RAP Content	Gyratory Angle (degrees)	GCI	GSI	Gg
SLM3.R1	30	1.54	0.9749	1.21	2289
SLM3.R2	30	1.43	0.9672	1.12	2743
SLM3.R3	30	1.56	0.9696	1.22	2609
SHM10.R1	100	2.20	0.9960	1.14	3559
SHM10.R2	100	2.80	0.9956	1.10	5229
SHM10.R3	100	3.07	0.9951	1.22	4157
SMM10.R1	100	2.05	0.9919	1.22	2744
SMM10.R2	100	1.88	0.9916	1.21	1813
SMM10.R3	100	2.15	0.9929	1.19	2416
SLM10.R1	100	1.59	0.9834	1.17	2429
SLM10.R2	100	1.77	0.9832	1.29	2120
SLM10.R3	100	1.74	0.9807	1.32	2093
STM3.R1	30	2.18	0.9874	1.67	2611
STM3.R2	30	2.16	0.9823	1.70	3175
STM3.R3	30	2.03	0.9820	1.62	3314
SRM3.R1	30	2.21	0.9880	1.71	1820
SRM3.R2	30	2.42	0.9879	1.84	1794
SRM3.R3	30	2.33	0.9903	1.81	1619
STL3.R1	30	2.25	0.9879	1.51	4109
STL3.R2	30	2.34	0.9873	1.66	4603
STL3.R3	30	2.18	0.9835	1.49	4424
SRL3.R1	30	2.49	0.9871	1.80	1972
SRL3.R2	30	2.62	0.9884	1.87	2083
SRL3.R3	30	2.39	0.9869	1.75	2206

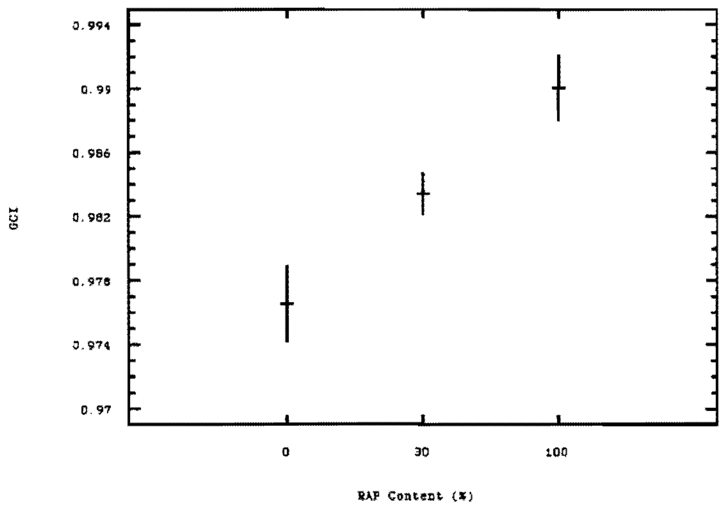

Figure 6. Compactibility Index For Materials Tested in the Experiment.

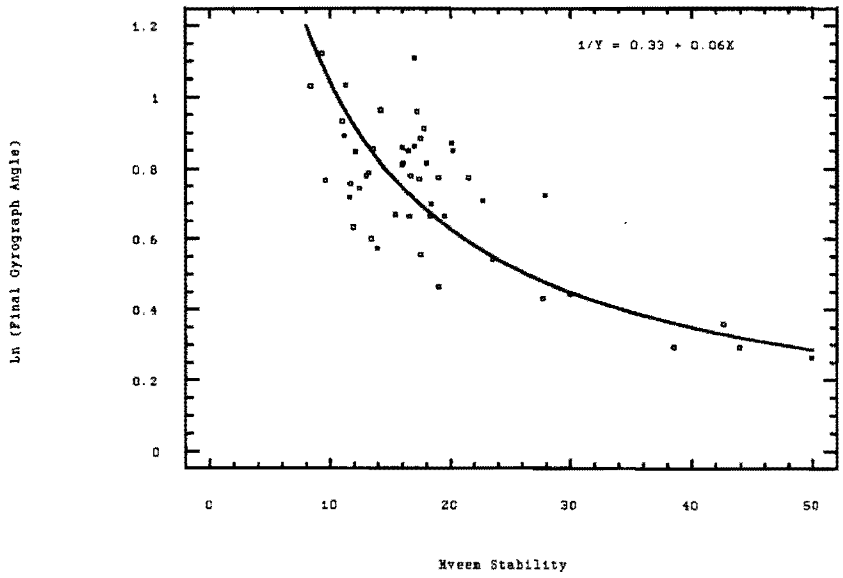

Figure 7. Ending Gyrograph Reading and Hveem Stability (All Mixes in the Experiment Combined).

These findings raise some question as to the prospects for rutting and flushing of the mix and the associated safety and performance degradation. Further testing was performed at TTI to determine specific gravity, air voids, and Hveem stability. The summary in Appendix B provides data on these tests. The results show that generally, the air voids decrease as the pressure increases from 517 kPa (75 psi) to 2068 kPa (300 psi) at a given temperature in the gyratory test. As Figure 8 illustrates, the finer graded 100% RAP shows a more rapid reduction in air voids than the larger, coarser CMHB (0% RAP). For the 100% RAP material at 250 gyrations, 2068 kPa (300 psi) and 121°C (250°F), the air void content is essentially zero (0.17%), while at 517 kPa (75 psi) and 1034 kPa (150 psi), the air voids were 2.8% and 1.4% for the same mix.

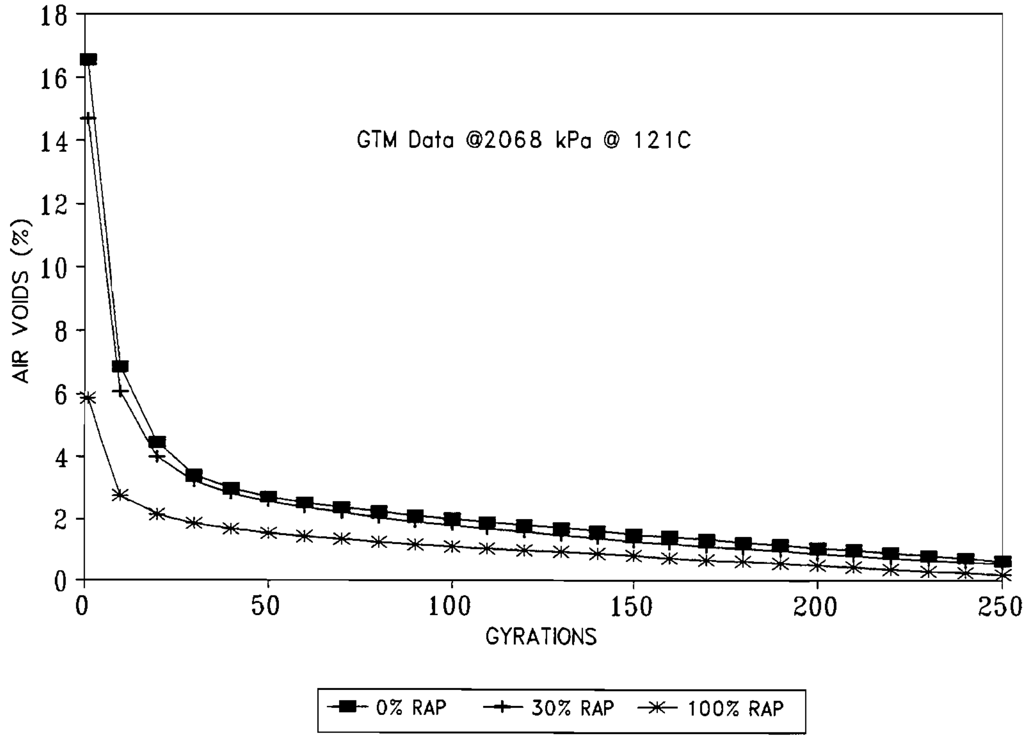

Figure 8. Air Voids vs. Gyratory Revolutions.

This trend indicates that the recycled CRM asphalt would be suitable for car and light truck traffic, but may show rutting or bleeding under heavy truck traffic. These results may be compared with the results shown by Roberts et al. (1991) and illustrated in Figure 9. The satisfactory performance of recycled CRM mixtures thus seems to depend on ensuring stone-on-stone contact. This might best be achieved by gap- and open-graded mixtures such as CMHB or SMA (Stone Matrix Asphalt) materials. The potential for segregation in this type of mixture seems to be alleviated by thick films of asphalt and by the presence of CRM, which helps prevent draindown of the binder.

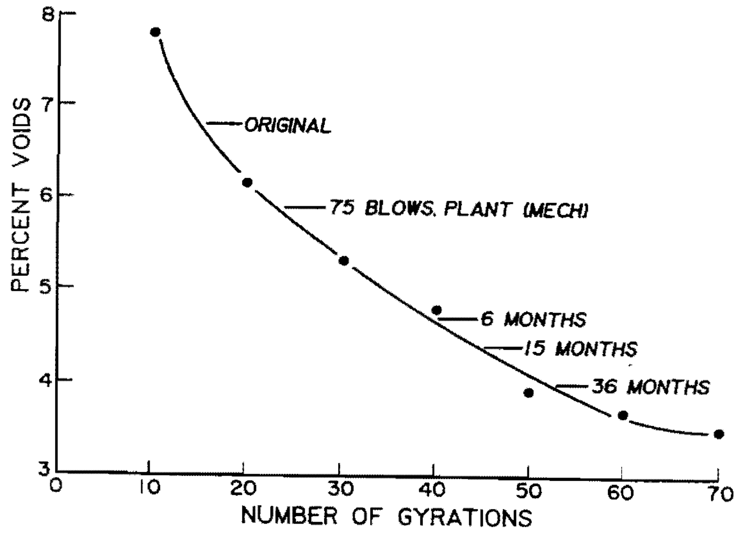

Figure 9. Air Voids vs. Gyrations at 689 kPa (100 psi) Compaction Level (after Roberts et al. 1991).

Permeability Characteristics

Water permeability of paving mixtures has always been important. Construction of zero permeability paving materials is elusive if not impossible. However, surfaces can be made with very low permeabilities and geometries that effectively make the pavement impermeable. At the other extreme, materials can be placed that have permeabilities high enough to act as drainage facilities and which do not support high pore pressures because there are relatively continuous paths through the pore structure to an exit. Between these two extremes lies the material that allows water into its internal structure and then proceeds to self-destruct under the cycling of pore pressure caused by traffic (augmented in some locations by damage associated with freeze-thaw cycles). The CMHB material actually has some of the characteristics of both of the extremes. The gradations used for the CMHB begin life with porous characteristics. Then the pore structure is filled with the fine aggregate fraction and rubberized binder.

Since there is no outlet for water on the San Antonio IH-10 test section because of the inlay technique, near zero permeability is necessary to prevent waterlogging in layers below the wearing course. An important factor affecting the permeability of asphalt pavements is the air void ratio, so it was of interest to define a relationship between air void ratio and permeability. Since there is no standard test procedure to test the permeability of bituminous mixtures, a procedure was developed as a modification of ASTM D 2434: Permeability of Granular Soils (Constant Head). The modified procedure prescribes a constant head method for dense mixes with less than 3% air voids, and a falling head method for open mixtures with higher air void contents.

The test method determines the coefficient of permeability for the laminar flow of water through the asphalt rubber. To ensure laminar flow under constant-head conditions, the ideal test conditions necessary are:

1) Continuity of flow with no volume change during a test,
2) Flow with the air voids saturated with water, and
3) Flow in the steady state with no changes in hydraulic gradient.

The permeameter used in this study was built in the TTI machine shop, with a minimum internal diameter of 5.0 cm (2.0 in.) and a maximum of 10.0 cm (4.0 in.), which is the size of standard molded specimens of asphalt concrete. Details of the test procedure are provided in Appendix A. From the determination of air void ratios on the mixes tested by WES, specimens with the highest and lowest air void ratios were selected for permeability tests.

Nine samples were tested for permeability in multiple runs. Five of the specimens were GTM samples, and the sixth was a field core cut from the original CRM mix laid on

IH-10. The range of air void ratios for all six specimens (0.292 to 3.049) was too small to realize any trends occurring, so it was necessary to mold new specimens having higher air void ratios. Five additional points were thus obtained from cores molded at 4.5%, 5.0%, 6.0%, 8.3%, and 10.0%. A multiple regression analysis was performed on the data, giving an R^2 value of 0.98 and the following linear relationship:

$$\log k = -2.31 + 0.41 \log voids + 1.95 \log gradient. \tag{1}$$

Figure 10 shows a graph of how permeability would vary with percentage air voids at a fixed gradient of 0.1. The results exhibit an expected overall relationship of permeability increasing with air void ratio, the k values for all eleven data points falling below 10^{-4} cm/s.

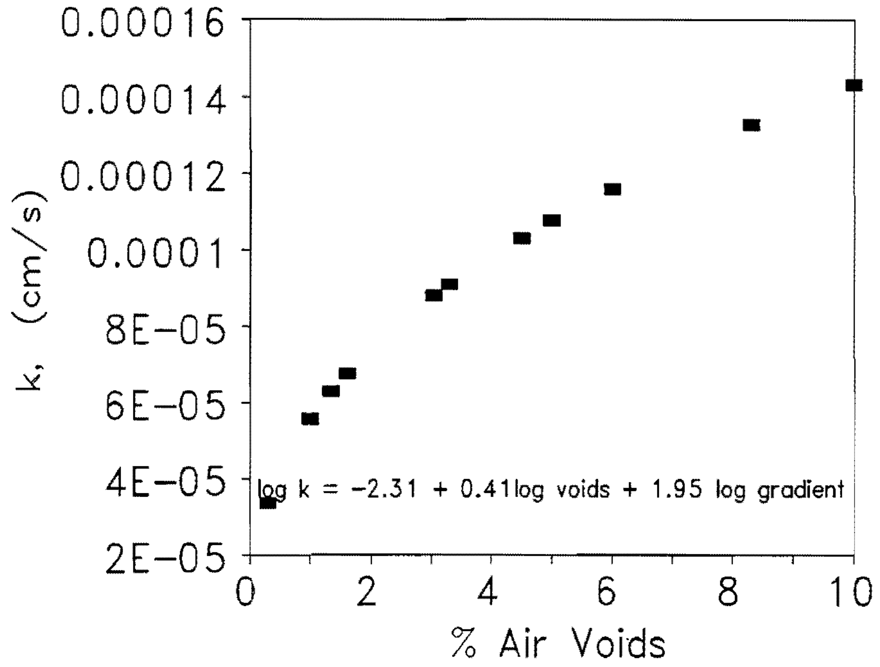

Figure 10. Permeability of CRM Asphalt vs. Air Voids (Gradient = 0.1, Falling Head Method).

While permeability and air void content are often assumed to be directly proportional, it must be stressed that a mixture with high air voids could easily have a low permeability, due to certain size dimensions of the individual voids and the lack of interconnection of the voids (Zube 1962). In Figure 10, the value of k appears to level off as air void content increases, suggesting that for the very open mixtures where the voids were connected at a maximum, the coefficient of permeability was limited by the maximum flow rate attainable by the apparatus. For the denser mixtures, the smaller the number of air voids present, the more the permeability depends on those few voids being well-connected. This explains why for two samples with comparable air void contents, the coefficient of permeability may be vastly different, as seen in Table 5. Apparently, the 30% RAP specimens have thicker asphalt films and thus fewer interconnected voids than those with 100% RAP at comparable air void contents. This reduction in permeability from first to second generation is favorable to the specific drainage needs of the recycle test section, which would be prone to moisture-assisted damage if the inlaid pavement were highly permeable.

Permeability and average velocity were plotted against average gradient. The relationship obtained appears to be linear where the gradient ranges from 0 to 0.4, as shown in Figure 11. This confirms laminar flow for low gradients and validates the use of Darcy's Law in this region.

Table 5. Dependence of k on Air Voids Interconnectivity.

% RAP in Sample	Air Voids (%)	k (cm/s)
100 (original I-10)	1.345	7.124×10^{-4}
30 (recycled)	0.996	1.048×10^{-4}
100 (original)	3.3	1.403×10^{-3}
30 (recycled)	3.049	5.537×10^{-5}

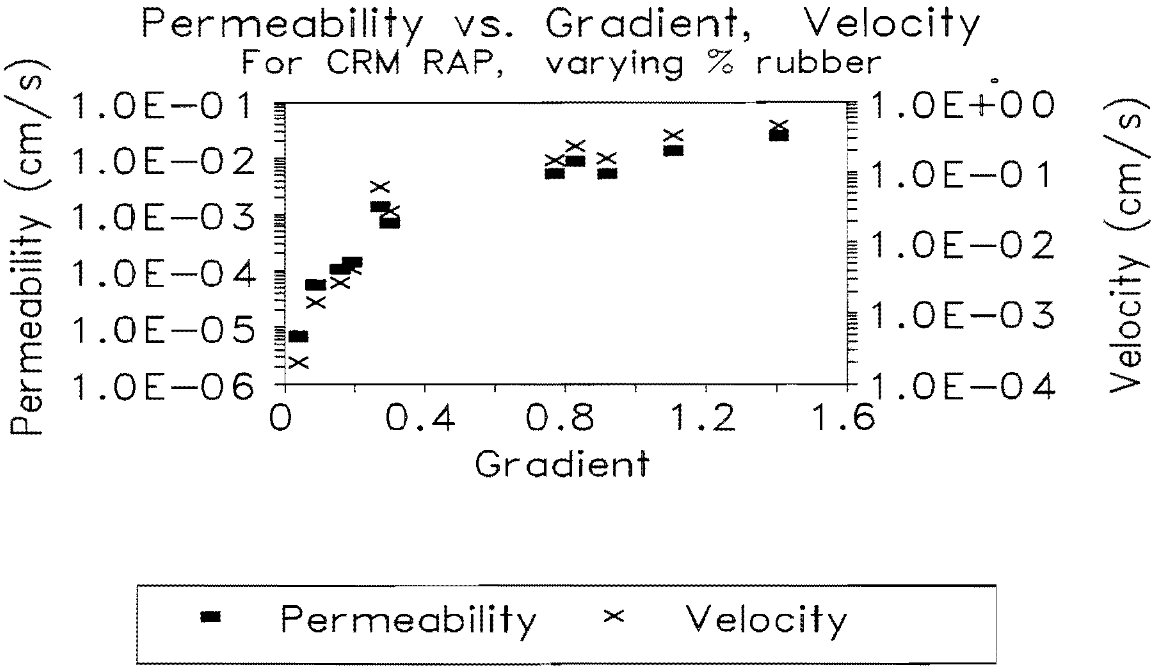

Figure 11. Permeability vs. Gradient and Velocity (Falling Head Method).

Creep and Compression Characteristics

CRM asphalt concrete is viscoelastic, possessing both an elastic component and a time-dependent viscous component that causes this material to flow gradually when subject to traffic. Creep testing measures time-dependent permanent deformation of CRM asphalt concrete. The unconfined compression strength is a measure of the maximum vertical load sustainable by a specimen before failure. The GTM compacted specimens which were tested for permeability were also tested for creep and compression strength in accordance with test methods TEX-231-F and AASHTO T-167. Table 6 provides a summary of the results from the creep test. More extensive data from creep testing of field cores and additional lab molded specimens can be found in the Appendix B. The variation of creep stiffness with CRM RAP content is shown in Figure 12.

Data from compression testing is given in Table 7, and Figure 13 shows the variation of compression strength with RAP content.

Table 6. Uniaxial Creep Test Results.

Specimen ID	CRM RAP Content (%)	Total Strain	Permanent Strain	Slope of SS portion (1/sec)	Creep Compliance (1/kPa)	Creep Stiffness Modulus S_{mix} (kPa)
SLMO.R3	0	6.649×10^{-4}	2.047×10^{-4}	2.100×10^{-8}	9.724×10^{-6}	102835
PAVE	0	1.367×10^{-3}	4.288×10^{-4}	5.200×10^{-8}	2.007×10^{-4}	49835
SML3.R1	30	2.317×10^{-3}	1.417×10^{-3}	1.640×10^{-7}	3.378×10^{-4}	29599
STL3.R3	30	1.845×10^{-3}	1.197×10^{-3}	1.490×10^{-7}	2.697×10^{-4}	37080
SRM3.R3	30	1.747×10^{-3}	1.022×10^{-3}	1.080×10^{-7}	2.549×10^{-4}	39231
SHM10.R3	100	3.315×10^{-3}	2.054×10^{-3}	2.260×10^{-7}	4.830×10^{-4}	20705

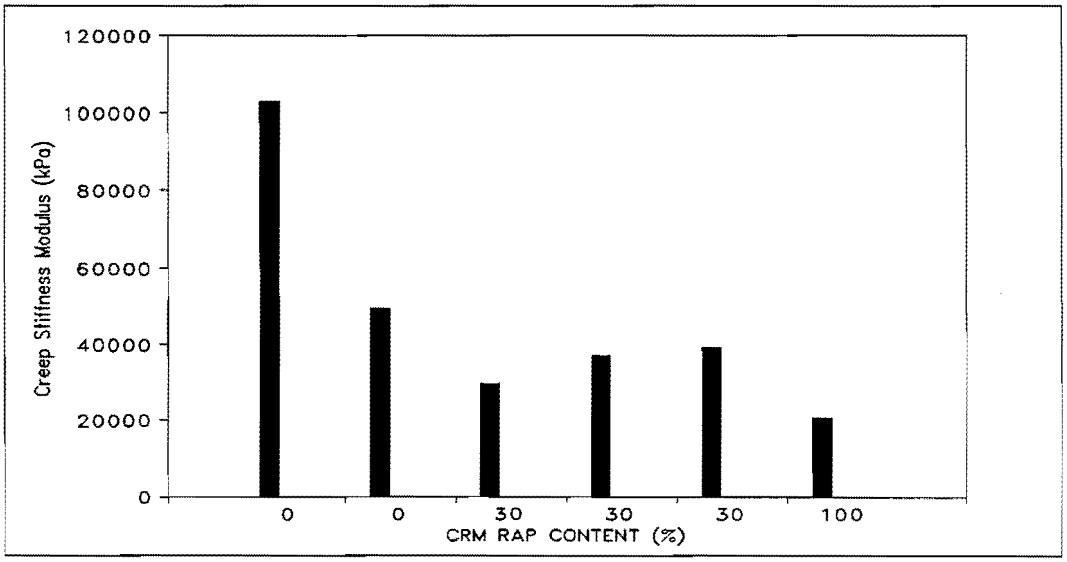

Figure 12. Variation of Creep Stiffness with Percentage CRM RAP.

Table 7. Compressive Strength Test Results.

Specimen ID	CRM RAP Content (%)	Compression Strength (kPa)
SLM0.R3	0	4806.2
PAVE	0	4302.5
SML3.R3	30	4983.0
STL3.R3	30	5840.0
SRM3.R3	30	4983.0
SPEC.3	30	4559.7
SHM10.R3	100	4488.5

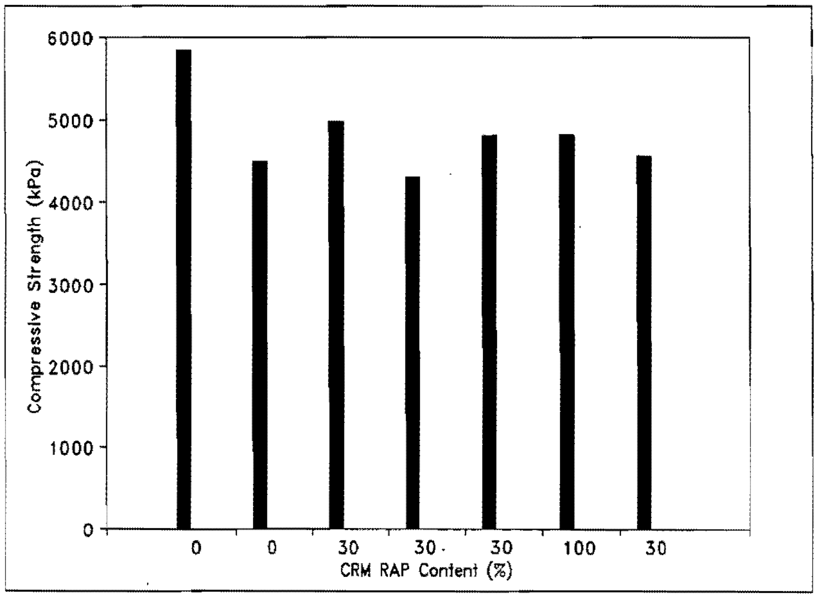

Figure 13. Variation of Compressive Strength with CRM RAP Content.

BINDER CHARACTERISTICS

Asphalt-rubber samples (asphalt meaning either asphalt or recycling agent) were aged under numerous conditions. The samples were POV-aged under 345 kPa (50 psia) of nitrogen and 93°C (200°F), under atmospheric air and 93°C (200°F), or under 2068 kPa (300 psia) of oxygen and 82°C (180°F) or 93°C (200°F). The samples were oven-aged with atmospheric air at either 49°C (120°F) or 163°C (325°F). To fully understand the difficulties of aging asphalt-rubber, a detailed explanation of research to date is required.

Nitrogen-Aged Samples

Asphalt-rubber samples were nitrogen-aged to determine if the samples cure, defined as an increase in viscosity without a corresponding increase in carbonyl area, when exposed to high temperatures. If the samples were found to cure at the high temperatures found in the POV, it would drastically complicate using the POV for an aging test. Diamond Shamrock AC-5, SHRP ABM-1, and blends of 10% and 18% or 20% of each asphalt were aged under 345 kPa (50 psia) of nitrogen and 93°C (200°F). The viscosity and the carbonyl area of the asphalts and the asphalt-rubber blends were determined to remain constant with aging time. This can be seen in Figure 14 and Figure 15, respectively. GPC analysis of the samples that were incubated under nitrogen also shows that no type of curing or aging occurs. This is shown in Figure 16 for the SHRP ABM-1 asphalt and in Figure 17 for the 18% by weight Rouse -0.425 mm mesh and SHRP ABM-1 asphalt-rubber. The asphalt acts like a solvent for the rubber, and because the rubber is crosslinked, it cannot dissolve or degrade, but can only swell. Apparently the swelling is caused by the rubber-asphalt bonds forming and/or replacing the strong secondary bonds of the rubber. However, the crosslinked polymers are not soluble because the solvent cannot overcome primary valence crosslinks, and thus the dissolution or degradation of the rubber does not occur. This contrasts with what occurs during mixing, with the material of the 20 to 25 retention time region increasing with mixing. This is shown in Figure 18 for a Fina AC-5 asphalt and Tire Gator -0.425 mm mesh mixing (curing) study. From the curing studies, it is known that a binder's viscosity increases and the carbonyl area remains constant when mixing under a

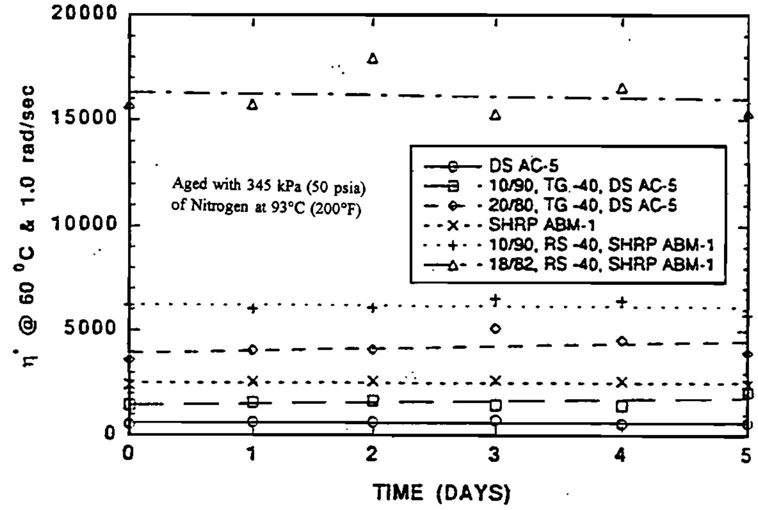

Figure 14. Viscosity Data of Nitrogen-Aged Samples.

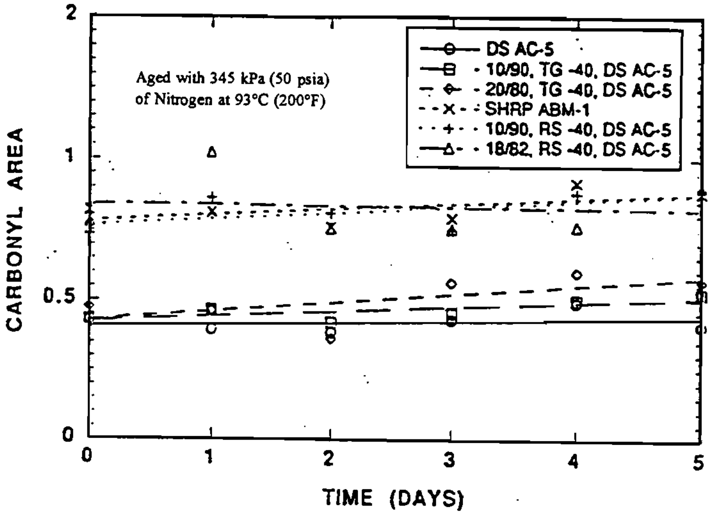

Figure 15. Carbonyl Area Data of Nitrogen-Aged Samples.

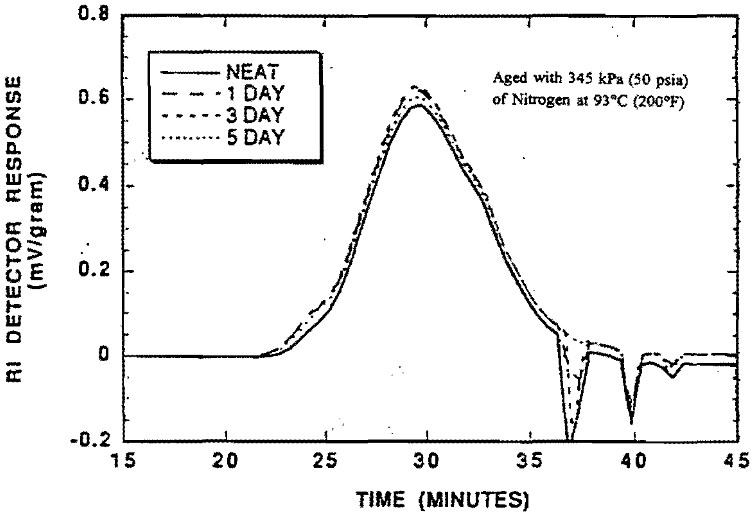

Figure 16. GPC Data of Nitrogen-Aged SHRP ABM-1.

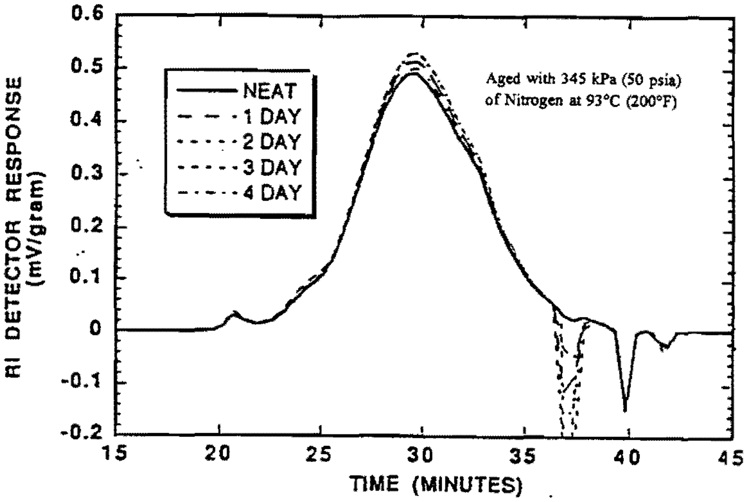

Figure 17. GPC Data of Nitrogen-Aged SHRP ABM-1 and Rouse-0.425 mm Mesh Blend.

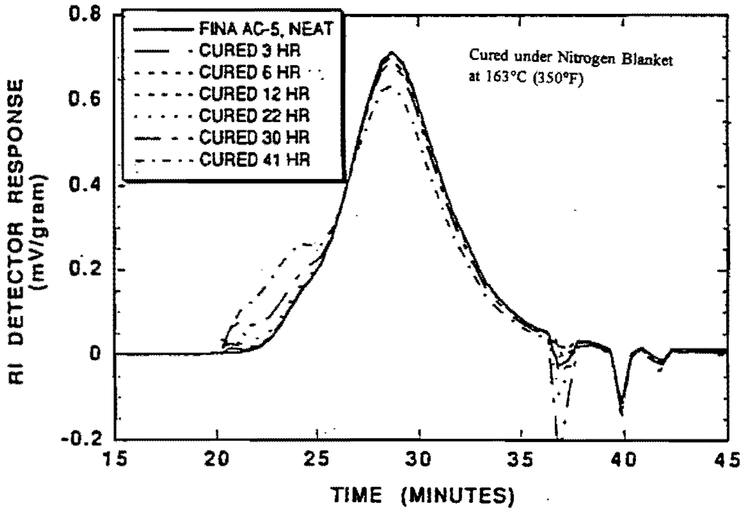

Figure 18. GPC Data of Curing Study (Fina AC-5, Tire Gator -0.425 mm Mesh).

nitrogen blanket at 177°C (350°F). It is theorized that the high mixing temperature enhances the rubber's degrading or reacting with the asphalt. At such a high mixing temperature, the entropy factor of the Gibbs free energy change equation, $\Delta G = \Delta H - \Delta TS$, dominates and thus makes the change in the Gibbs free energy negative, increasing the spontaneity of the reaction. Additionally, the mixing apparently supplies the continual dispersement that helps the rubber and the asphalt to chemically bond.

Oven or POV-Aged Samples

GPC Analysis

Asphalt-rubber samples were aged under various combinations of pressure and temperature. Decomposition of the rubber into the asphalt part of the asphalt-rubber binder occurs at all combinations of pressure and temperature. This is shown in Figures 19-21 for SHRP ABM-1 asphalt-rubber blends, with Figure 19 representing high temperature and high oxygen pressure conditions, Figure 20 representing high temperature and atmospheric

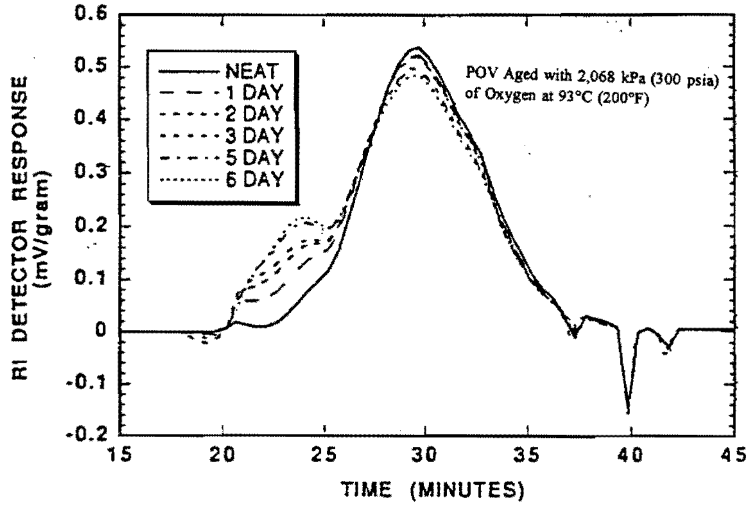

Figure 19. GPC Data of SHRP ABM-1 and Tire Gator -0.425 mm Mesh Blend POV-Aged at 93°C with 2,068 kPa of Oxygen.

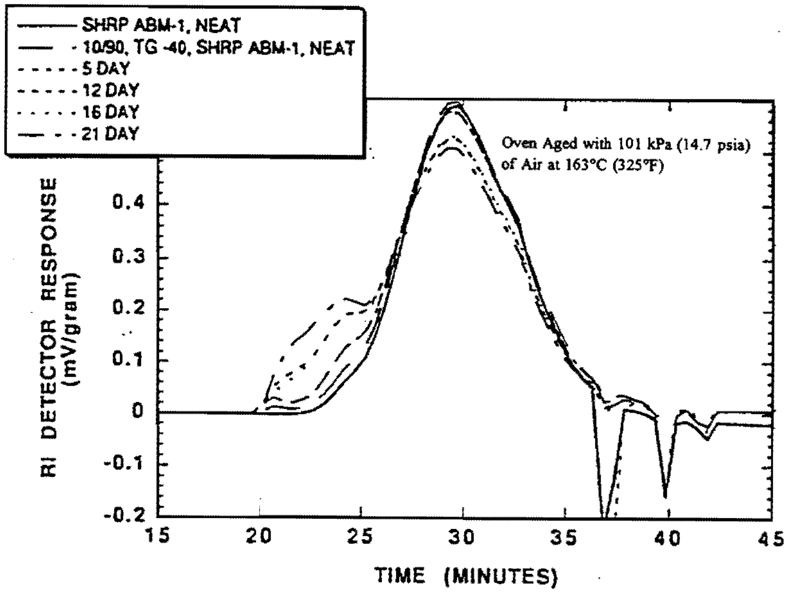

Figure 20. GPC Data of SHRP ABM-1 and Tire Gator -0.425 mm Mesh Blend Oven-Aged at 163°C with 101 kPa of Air.

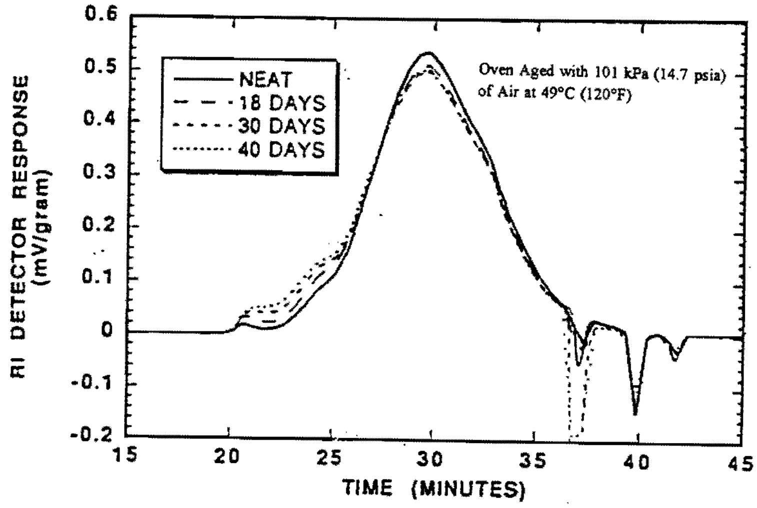

Figure 21. GPC Data of SHRP ABM-1 and Rouse -0.425 mm Mesh Blend Oven-Aged at 49°C with 101 kPa of Air.

oxygen pressure conditions, and Figure 21 representing road conditions of summer road temperature and atmospheric oxygen pressure. It is theorized that the decomposition of the rubber into the asphalt is caused by oxidation involving chain scission of the completely substituted carbon. This contrasts with what occurs in the mixer, where the rubber degrades by thermal energy, and with what occurs in the POV under nitrogen, where the rubber does not degrade, apparently because of the lack of oxygen.

Furthermore, the GPC analysis showed that the interaction of the rubber and the asphalt depends largely upon the composition of the asphalt. Figures 19, 20, and 21 are data of the decomposition of the rubber into the asphalt, SHRP ABM-1, that has significant percentages of each of the four asphalt groups: asphaltenes, saturates, napthene aromatics, and polar aromatics. Figures 22 and 23 are data of the POV-aging of SHRP ABM-1 Fraction and rubber/SHRP ABM-1 Fraction, respectively. The SHRP ABM-1 Fraction is a supercritical fraction of the SHRP ABM-1 asphalt with the fraction being nearly saturate and asphaltene-free. Figure 22 shows that the SHRP ABM-1 Fraction produces only small

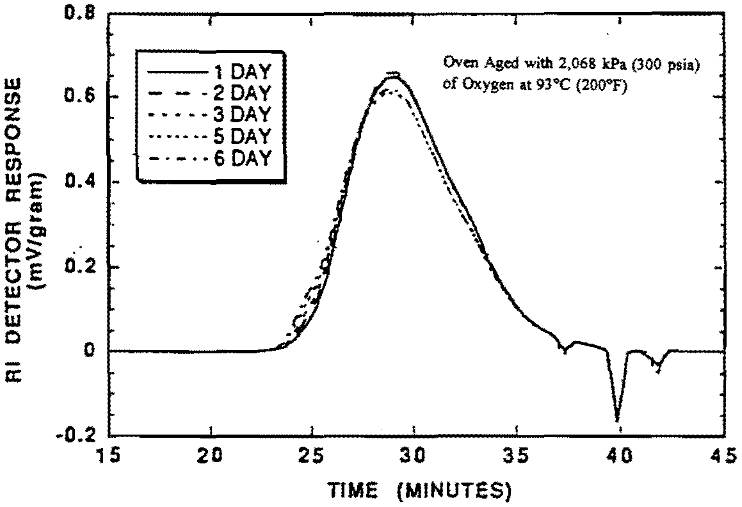

Figure 22. SHRP ABM-1 Fraction POV-Aged at 93°C with 2,068 kPa of Oxygen.

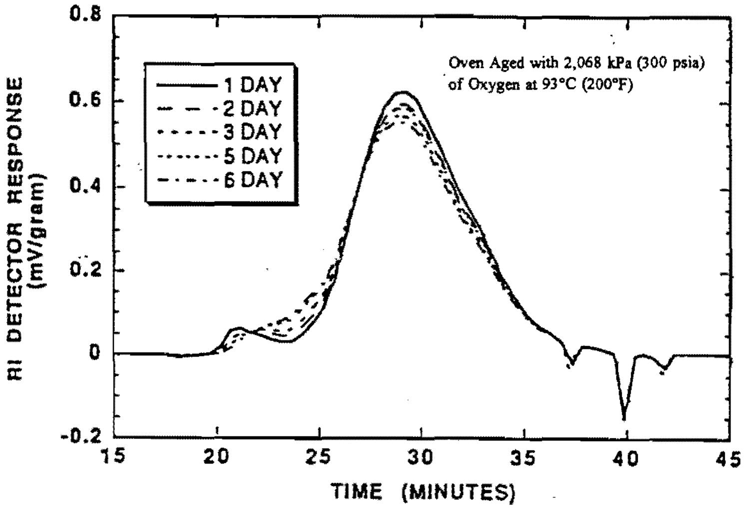

Figure 23. GPC Data of SHRP ABM-1 Fraction and Tire Gator -0.425 mm Mesh Blend POV-Aged at 93°C with 2,068 kPa of Oxygen.

quantities of asphaltenes with aging. Figure 23 shows that the rubber decomposed into the binder during curing and was further decomposed during aging, with the mass from the neat binder curing peak being systematically moved into the longer retention time region with aging time. Note the lack of influx of mass from the rubber with aging time as with the whole asphalt/rubber binder. It is theorized that the lack of rubber decomposition during the aging stage is caused by the absence of asphaltenes in the ABM-1 Fraction. Additionally, Tire Gator -0.425 mm mesh rubber was cured with SUN Hydrolene 125, a recycling agent containing manufacturer reported values of 0.1% asphaltenes, 13.5% polar aromatics, 70.9% napthene aromatics, and 15.5% saturates. Figure 24 shows that the rubber in the SUN 125/rubber blend did not decompose upon aging, supporting the earlier theory that asphaltenes must be present if the rubber is to be compatible with or soluble in the asphalt.

Hardening Susceptibilities

Previous work has shown that asphalt hardening susceptibilities, the slope of a log viscosity versus corrected carbonyl area plot, are independent of POV temperature. However, this work shows that asphalt-rubber hardening susceptibilities are dependent on POV temperature. Figure 25 shows this for the SHRP ABM-1 and Tire Gator -2 mm mesh blend. Similar results were obtained for all other POV-aged blends. It can be mathematically proven that for the hardening susceptibility of the asphalt to be independent of temperature, the activation energy of each of the components of the asphalt -- asphaltenes, polar aromatics, napthene aromatics, and saturates -- must be equal. In the case of the rubber and asphalt blend, with the hardening susceptibilities being dependent upon the temperature, the rubber and the asphalt must have different activation energies.

The addition of rubber improves the hardening susceptibilities of binders at all POV and oven conditions tested. This is shown for ABM-1 and ABM-1 blends in Figures 26 (at 163°C and 101 kPa air), 27 (at 82°C and 2,068 kPa oxygen), and 28 (at 49°C and 101 kPa air). Furthermore, the higher the rubber content, the better the improvement in the hardening susceptibilities. This improvement with rubber content can be observed in Figure 27 and Figure 28. It is theorized that the rubber has a lower activation energy than the asphalt and is thus the sacrificial anode that is more susceptible to oxidation. Visual inspection of the

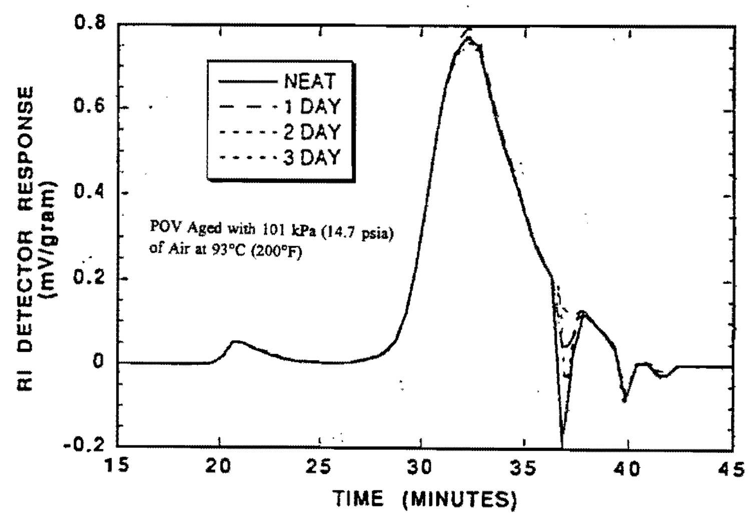

Figure 24. GPC Data of SUN 125 and Tire Gator-40 Mesh Blend POV-Aged at 93°C with 101 kPa of Air.

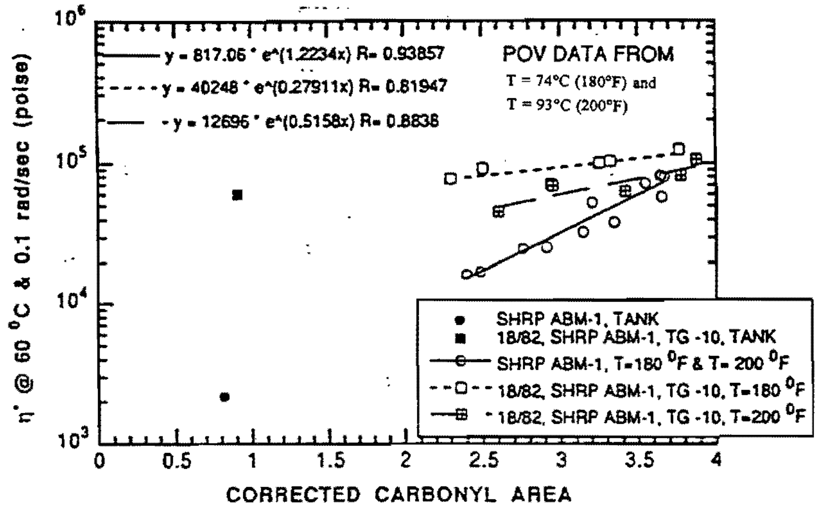

Figure 25. Change of Hardening Susceptibilities with POV Temperature for SHRP ABM-1 Blends.

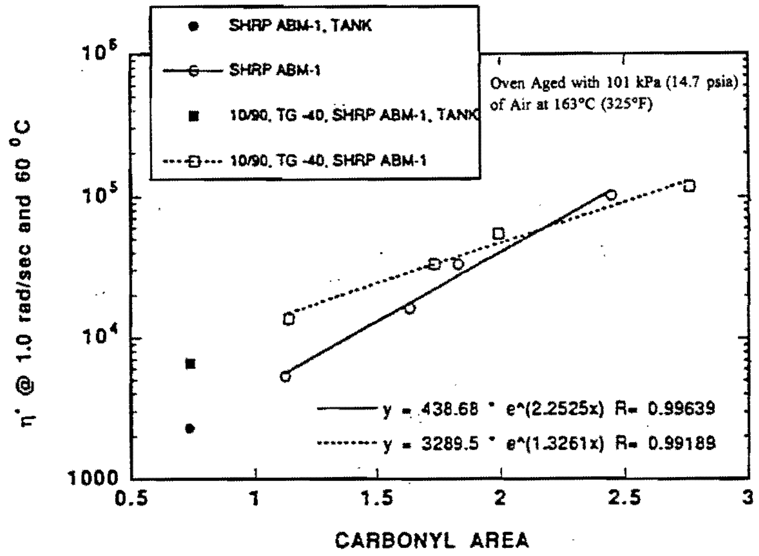

Figure 26. Hardening Susceptibilities of SHRP ABM-1 and Blends Oven-Aged at 163°C with 101 kPa of Air.

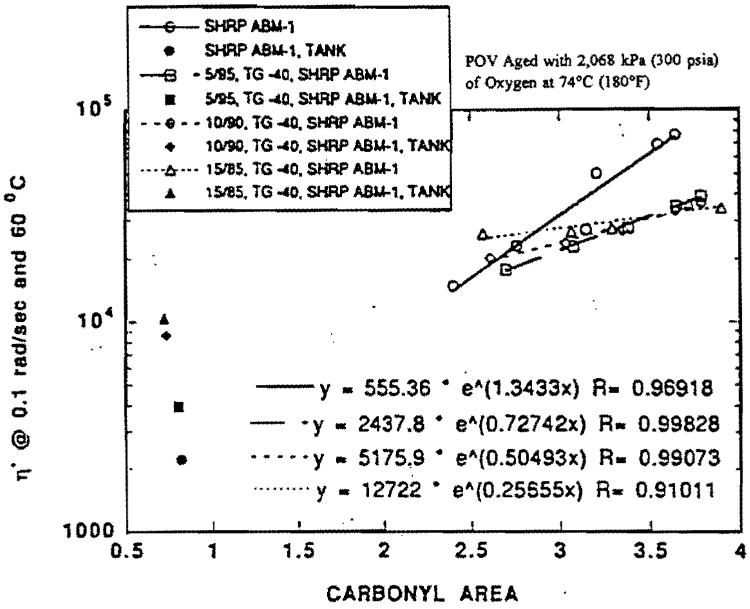

Figure 27. Hardening Susceptibilities of SHRP ABM-1 and Blends POV-Aged at 82°C with 2,068 kPa of Oxygen.

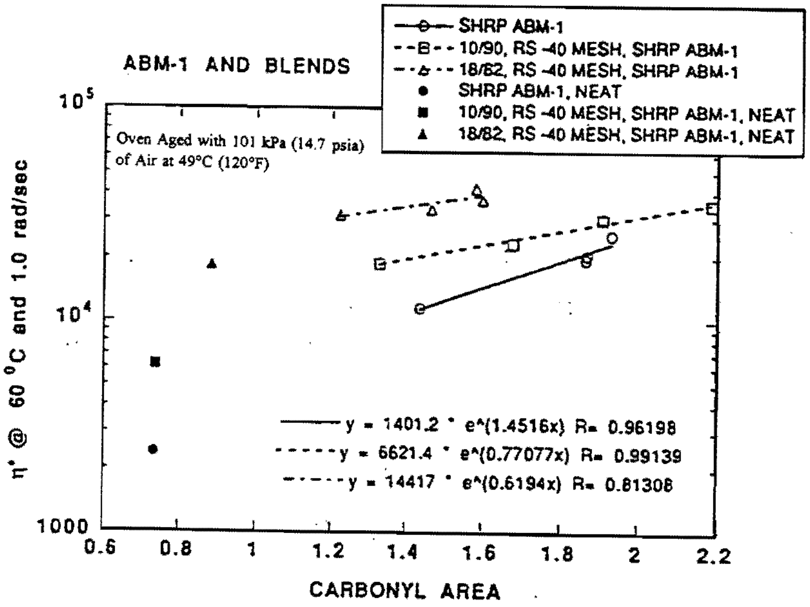

Figure 28. Hardening Susceptibilities of SHRP ABM-1 and Blends Oven-Aged at 49°C with 101 kPa of Air.

POV-aged samples shows that the asphalt-rubber binders maintain flexibility; whereas, the straight asphalts become brittle and glass-like.

Aging Rates

The aging rate, the slope of a carbonyl area versus time plot, was found to increase with rubber content. This is shown in Figure 29 for the Diamond Shamrock AC-5 blends and in Figure 30 for the ABM-1 blends. Initially, the antioxidants in the rubber help inhibit oxidation, with the carbonyl area of the blends being less than the carbonyl area of the straight asphalt. After the antioxidants are exhausted, the rubber acts as a catalyst to speed aging, with the carbonyl area of the blends being greater than the carbonyl area of the straight asphalt. However, in the overall aging picture, this is negligible since the hardening susceptibility of a blend is much lower than the hardening susceptibility of the corresponding asphalt. Thus, the hardening rate, the slope of a log viscosity versus time plot, for an asphalt-rubber is lower than the hardening rate for the corresponding asphalt. This is shown in Figure 31 for SHRP ABM-1 and blends.

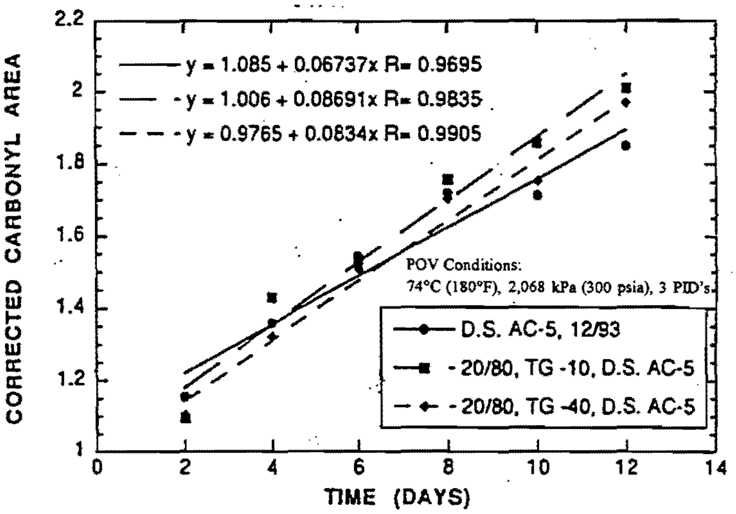

Figure 29. Aging Rates of Diamond Shamrock AC-5 and Blends.

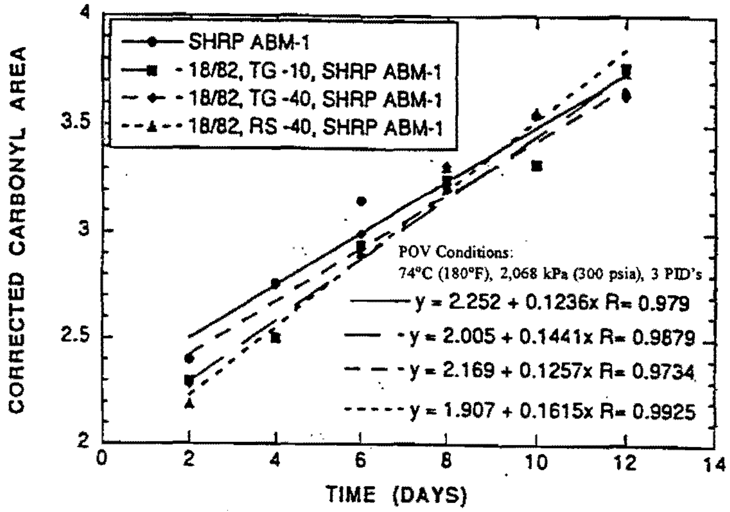

Figure 30. Aging Rates of SHRP ABM-1 and Blends.

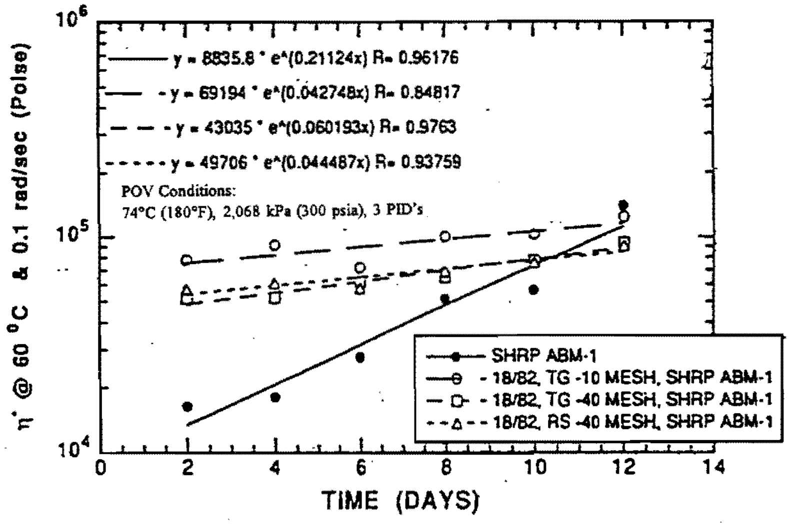

Figure 31. Hardening Rates of SHRP ABM-1 and Blends.

III. CONCLUSIONS

ENVIRONMENTAL

Environmental testing showed that there was very little difference between the emissions from the limited number of CRM and standard asphalt plants tested in this study. For the recycled CRM mixture, VOC emissions were lower than the range for standard HMA. Variations in the conditions of a hot mix operation sometimes confound the effect of CRM on emission rates. Trace metals, volatile organics, and semivolatile organics may be leached from asphalt rubber, but at levels too low to be environmentally significant or hazardous under current guidelines.

DESIGN AND CONSTRUCTION PRACTICE

Mix Design

A proposed mix design procedure is presented in Appendix C. The mix should maintain stone-on-stone contact and adequate space to accommodate the rubber and the RAP; and sufficient rubber and fines must be present to prevent draindown. If the aggregate and aggregate gradation are good, if the binder can be rejuvenated, and if the RAP is not too wet, it may be be possible to recycle 100% CRM RAP mixtures.

Pavement Design

The use of higher permeability materials in an inlay application in which no drainage outlets exist and in which the inlaid material is surrounded by significantly lower permeability materials will result in the retention of water in the inlaid section and/or infiltration into lower layers in the pavement.

Construction Practice

The surface texture remaining after a cold milling operation is excellent. A plant mix seal prior to overlay on such a surface seems unnecessary. A moderate to heavy tack coat would seem to be adequate on the milled surface. If vibratory rolling is used, at least two coverages must be applied. At 30% CRM RAP content, rubber-tired pneumatic rollers perform reasonably well with standard release agents. Even though CRM mixtures often cool more rapidly than standard mixtures in the laboratory, long (on the order of 1 hour) hauls of hot recycled materials from the plant to the laydown site do not result in unacceptable temperature reduction during typical warm weather construction season operations in Texas.

Plant recycling appears to be the most viable option for recycling hot mix at present. Counter-flow drums appear to work well in this application. Hot in-place recycling/repaving without environmental emission control systems on the recycling train appears to be unacceptable in terms of opacity. The Pyrotech equipment might be successful in a hot in-place recycling operation. Cold in-place recycling as a base course is a viable option.

PERFORMANCE

Evaluation of mix designs used in the original IH-10 overlay, the Loop 1604 test section, and the IH-10 recycling showed that a CMHB or SMA type mix design gives a higher durability with acceptable Hveem stability than dense-graded mixes. Such mixes had +2 mm fractions of around 75% and asphalt contents of 5% - 6%.

Results from creep testing showed that the permanent strain increases with increasing CRM RAP content, and had no discernible relationship with air void content. An increase in permanent strain implies a greater susceptibility of CRM asphalt concrete to permanent deformation. Creep compliance increased with increasing CRM RAP content. No such trend was seen with increasing air void content. Compression strengths decreased as the CRM RAP content increased, and no trend was observed with air void content. However, these reductions in performance indicators with increasing CRM RAP content are confounded with mix design issues and may have more to do with gradations than with the rubber or even the RAP content.

The values for GSI and GCI showed that the CRM mixtures have low gyratory stabilities and are easy to compact. A relationship exists between the Hveem stability of a CRM specimen and its final gyrograph angle on the GTM. Thus a regression equation can be used to estimate gyratory strain equivalents when only the standard Hveem stabilometer is available for testing. No relationship was found between the Hveem stability versus air voids and creep. The authors do not agree with the philosophy of reducing the specification requirements for Hveem stability with CRM RAP mixes. Proper mix design does not seem to require such questionable changes for material acceptance.

PERMEABILITY

Results from permeability testing determined the permeabilities of CRM mixtures to be low, ranging from 6.7×10^{-6} to 2.53×10^{-2} cm/s. As the interconnection of the air voids in the pore structure approaches a maximum, the permeability of the CRM mixture is limited by the maximum flow rate attainable by the apparatus. Permeability decreases with increasing asphalt film thickness and decreasing air voids. It follows that a recycled mixture used in an overlay or inlay will not worsen durability by allowing any more percolation than was previously experienced, provided that the design of the mixture incorporates thick films, stone-on-stone contact, and appropriate air void contents.

BINDER PROPERTIES

The addition of rubber improves the oxidative properties of a binder relative to the base asphalt. The rubber apparently has a lower activation energy than the base asphalt and thus reacts more rapidly than the base asphalt. This is good, since the product of the oxygen/rubber reaction is less detrimental to the binder than the product of the oxygen/asphalt reaction. Furthermore, the extent of the benefits received by the addition of rubber is dependent upon the composition of the asphalt. However, the difference in activation energies makes the hardening susceptibility dependent upon the temperature, and thus material aged at high temperatures, 77+ °C (170+ °F), is not representative of material aged at road conditions. This complicates the process of aging sufficient quantities of asphalt-rubber to establish guidelines for the selection of recycling agents for recycling old asphalt-rubber pavements. Additionally, because the oxidation interaction of the asphalt and the rubber is so dependent upon the composition of the asphalt, the composition of the asphalt is the most important variable in the study of the oxidative aging and thus the recycling of asphalt-rubber binder. Furthermore, the dissolution of the rubber into the asphalt base with oxidation complicates the process of selecting the type and amount of recycling agent.

IV. RECOMMENDATIONS

ENVIRONMENTAL

If comparisons are to be made between plants for air emissions from hot mix operations, operating conditions should be equalized during sampling to keep plant production rates and temperatures consistent. Also, sampling operations should be scheduled within days of each other, rather than weeks, in order to minimize variability due to varying ambient conditions, and fuel and feedstock properties.

For a satisfactory statistical analysis of levels of air emissions and leachates, the scale of the sampling operation should be increased to examine more plants, more temperature conditions, and more than three replications for each sampling condition.

PERFORMANCE

The mix design procedure given in Appendix C is recommended to improve performance. Alternatively, a combination of existing TxDOT CMHB and CRM design procedures could be used.

PERMEABILITY

In determining the permeability of CRM mixtures, we observed that the values determined early in the study for the coefficient of permeability were much lower than those presented in the literature. The constant head apparatus used in the early stages had a limiting flow rate, probably due to narrow orificies or due to a contaminated porous disk at the bottom of the permeameter. These limitations could imply other confounders such as a wall friction factor, or even flow that is turbulent instead of laminar. To minimize these possibilities prior to running tests, trial runs should be made to ensure that the permeability apparatus used will accommodate the range of permeabilities to be determined.

The permeability test should account for three key mechanisms. First, it is thought that the pore pressures generated in a laboratory permeability test are much greater than those ever experienced in the field under traffic, even just after a rainfall. Second, the degree of saturation in the lab is essentially 100% percent. This is not usually attained in the field. Third, it is possible that the confinement of the sample in the permeameter does not accurately simulate confinement in situ. Further studies may examine the correlation between pore pressure, degree of saturation, and confinement generated in the field versus the lab.

It must be noted that the test specimens that had greater than 3% air voids were fabricated at TTI, separate from the GTM specimens which had lower air void contents. To confirm the relationship between permeability and air voids found in this study, it would be valuable to test samples that were all identically fabricated, at one time. Because the test procedure is still somewhat experimental, future permeability testing should include several replications.

It is recommended that a database be developed as more recycling takes place. This database should include permeability and air void measurements along with film thickness determinations so that permeabilities can be predicted on the basis of surrogate tests.

DESIGN AND CONSTRUCTION

It is recommended that inlay designs be used only after mandatory drainage evaluations and that appropriate actions be based on those evaluations.

The method of bonding the recycled material to the existing surface should be evaluated (e.g., tack coat versus underseal versus membrane/interlayer).

Standard methods for cold in-place recycling as a base course are recommended. Hot in-place recycling should only be done with equipment trains having effective emission control systems on them (e.g., Pyrotech). Standard plants with effective emission control devices appear to be adequate when incorporating up to 30% CRM RAP in the mixture. Counterflow drums may be slightly more environmentally friendly. The Rapmaster system should be evaluated.

Standard compaction equipment (including pneumatic rubber-tired rollers) may be used. Coverages by vibratory drum rollers should never be less than two, and should be an even number if possible. While rubber-tired equipment operated well at 30% CRM RAP in the mix, it is unknown at what CRM RAP percentage tire pickup will become a problem. This should be evaluated and various release agents should be formulated and tried on jobs with higher CRM RAP percentages. For environmental reasons, cold or very low heat milling is preferred over hot milling.

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APPENDIX A - TEST PROCEDURES USED IN THIS STUDY

CORPS OF ENGINEERS GYRATORY TEST

ASTM D3387 is used with asphalt mixtures, or tar and aggregate. The test method employs two separate modes of operation of the GTM, namely a) the fixed roller mode and b) the GTM oil-filled roller mode. The first mode is employed in testing for compaction and strain indices only, while the latter is employed in testing for strength properties as well as compaction and strain indices. The following definitions are key to gaining a full understanding of the test procedure and its measurements:

gyrograph - a recording of shear strain experienced by the bituminous mixture during the compaction test.

gyratory angle - a measure of the magnitude of gyratory strain, where
h_o = initial gyratory angle or shear strain, and
h_i, h_{max} = minimum and maximum gyratory angles or shear strains.

gyratory stability index (GSI) - the ratio of h_{max} to h_i.

gyratory compactibility index (GCI) - the ratio of the unit mass (total mix) at 30 revolutions of the GTM to the unit mass (total mix) at 60 revolutions of the GTM.

gyratory shear strength (S_G) - the shear resistance of a specimen.

gyratory shear factor (GSF) - the ratio of the measured gyratory shear strength to the approximate theoretical maximum induced shear stress.

The use of this method for guidance in the selection of the optimum bitumen content is applicable to mixtures that are susceptible to the development of excess pore pressure when the voids become overfilled with bitumen, or asphalt binder. Such is the case for the asphalt rubber tested. A GSI of greater than one indicates a progressive increase in plasticity during densification. Thus an increase in this index indicates an excessive asphalt binder content for the compaction pressure employed and predicts instability of the asphalt rubber for the loading employed.

The three design mixtures tested at WES were as follows: a) 100% RAP, which is actually the original CRM material that was placed on IH-10 in 1992; b) 30% RAP, obtained from milling IH-10, and 70% conventional asphalt mixture - this mix was actually placed on the Loop 1604 recycle test section; and c) 0% RAP, a virgin CMHB asphalt mixture. The specimens used were molded cylindrical cores, with a 100 mm (4 in.) diameter, and an approximate height of 6.25 cm (2.5 in.).

An assembly drawing of the GTM is shown in Figure A-1. The test procedure is presented here in abbreviated detail. Using the oil-filled roller, h_0 is set at 1°, and is adjusted using a trial batch of mix. The GTM heater is set at 60°C (140°F) at least 15 minutes before starting the compaction test, and the mold and base plate are preheated. The asphalt rubber mixture is poured into the mold, with paper disks in the bottom of the mold and on top of it to prevent adhesion of the specimen to the end plates. Then the mold is placed in the machine and a vertical pressure applied just sufficient to retain the specimen while the front of the mold chuck is securely tightened. The vertical pressure is now increased to the full compaction test pressure.

The gyrograph recorder pin is brought in contact, and the roller carriage is actuated until 29 revolutions have been applied. At the completion of 29 revolutions, the carriage is stopped and the specimen height and roller pressure readings at positions 1, 3, and 4 are recorded, thus completing 30 revolutions. Then, additional revolutions are applied until a total of 59 is reached, readings are taken once again, and, thus, 60 revolutions are completed. In this study, automated recording was used out to 250 revolutions.

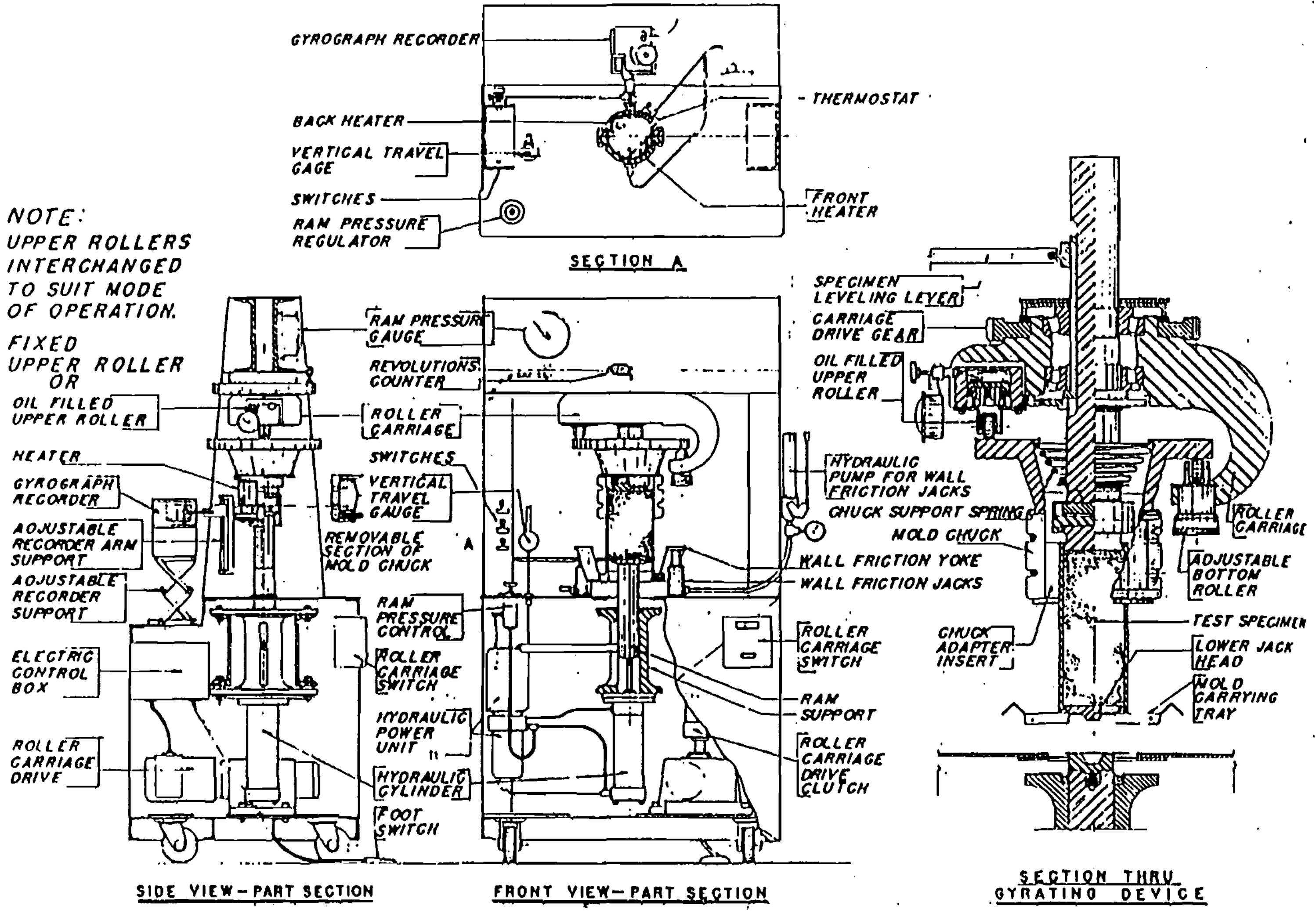

Figure A-1. Assembly Diagram of GTM.

WATER PERMEABILITY OF CRM ASPHALT MIXTURES

The permeability of virgin and recycled CRM was investigated at TTI. Since there is no outlet for water on the IH-10 test section because of the inlay technique, near zero permeability is necessary to prevent waterlogging in layers below the wearing course. An important factor affecting the drainage of asphalt pavements is the air void ratio, so it was of interest to observe how this factor affects permeability. While there is no standard test procedure in place to test permeability of bituminous mixtures, the procedure developed is in accordance with ASTM D2434.

Permeability of Granular Soils (Constant Head)

This test method determines the coefficient of permeability for the laminar flow of water through the asphalt rubber. To ensure laminar flow under constant-head conditions, the ideal test conditions are:

1) Continuity of flow with no volume change during a test,
2) Flow with the air voids saturated with water, and
3) Flow in the steady state with no changes in hydraulic gradient.

According to Darcy's equation for laminar flow at steady state, the coefficient of permeability, k, is determined by the relationship

$$k = QL/Ah$$

where Q is the flow rate through the area of the top of the sample (cm/s), L is the height of the specimen (cm), A is the cross-sectional area of the sample (cm^2), and h/L, or i, is the gradient (Crockford and Yang 1990).

METHOD I: DETERMINATION OF THE PERMEABILITY OF CRUMB RUBBER MODIFIED ASPHALT (Constant Head)

Scope

This test method describes the procedure for the determination of the coefficient of permeability, k, of crumb rubber modified asphalt mixtures. A constant head method is employed for the laminar flow of water through this material, with the use of a standard Texas triaxial cell apparatus, which utilizes a kerosene-water interface to supply both confining pressures and pore pressures. This procedure is applicable to mixtures with no more than 3% air voids.

Referenced Documents

1. ASTM Standard: D2434-68 Standard Test Method for Permeability of Granular Soils (Constant Head).
2. ASTM Standard: D4767-88 Standard Test Method for Consolidated-Undrained Triaxial Compression Test on Cohesive Soils.

Apparatus

Water Deaeration Device, to remove air from the water used to saturate the specimen.

Vacuum Pump, for evacuating air and saturating specimen under full vacuum.

Permeameter, consisting of:

- a 102 mm (4 in.) diameter triaxial cell fitted at the bottom with a mounting base containing a porous disk of a permeability greater than that of the specimen (> 10^{-4} cm/s), but with openings small enough to prevent movement of particles;

- manometer outlets for measuring head loss, h, over specimen length l;
- a top platen of a 102 mm diameter fitted with a porous disk in the face in contact with the specimen, and a tube fitting threaded into the top face.

Constant Head Filter Tank, which supplies flow to the sample and is regulated either manually or by a vacuum regulator.

Volume Change Measurement Device, a burette containing a kerosene-water interface to measure volume of water entering or leaving the specimen.

Rubber Membrane and Compatible O-rings, used to encase the specimen and seal off lateral flow into or out of the specimen. Two thickness of membranes are recommended to allow for double sheathing.

Test Specimen Preparation

Specimen Size and Form

Specimens shall be cylindrical CRM cores molded by a compaction method or drilled from the field, whose air void content has been previously determined. The standard compacted mold will have a 102 mm diameter, and a height of 63.5 mm (2.5 in.). Record the exact height of the specimen.

Saturation of Specimen

The purpose of saturating the specimen is to remove air in the voids in the material, and to fill them with water as completely as possible before the test is run. This is achieved by de-airing water in the deaeration device and using this water to submerge the specimen and porous stones in a dessicator flask. The air-tight flask is connected to the vacuum pump, and a partial vacuum is applied for about 20 minutes, or at least until vigorous bubbling of air has visibly ceased. The flask is then vented, and its contents are allowed to saturate for a 24-hour period.

Mounting of Specimen

The specimen should be checked to ensure that the top and bottom faces are not rough or ravelling, to ensure close contact with the porous stones. If the face edges have any pits, the pits should be sealed with molding clay worked to restore cylindrical form to the sample. Such seals are suitable on the circumference only, and should not be used on the cross-sectional faces of the specimen as they inhibit permeability. Cap the top and bottom of the specimen with 10.2-cm porous stones, and roll on a thin rubber membrane of sufficient length to encase the specimen, the 2 porous stones, the mounting base, and the top platen when stacked in a column. This first membrane is necessary to protect an outer membrane from piercing due to rough edges in the specimen, since even the slightest pressure leak will void the test. Slip the thicker membrane over the first one, mount the stack on the base in the triaxial cell, and cover it with the top platen. Seal the entire stack with rubber O-rings at the base and top platen. With the specimen in place, fill the cell with water and seal it.

Procedure (See diagram of test apparatus (Fig A-2)).

A. Filling Triaxial Cell

1. Connect the triaxial cell to the constant head filter tank and burette apparatus.
2. In the following order, open valves E (main water supply), F, C, C_2, on the panel board, and T_1, T_5, and T_6 on the triaxial cell. Water should start flowing into the triaxial cell.
3. Control the flow of water with T_1 so no air bubbles are entrapped in the water as the cell fills. If it is necessary to lower the water level to remove air bubbles, close T_1 and open T_4.

4. When sufficient water flows out of T_5 and T_6 to carry out the air entrapped beneath the cap of the cell, close T_1, then close T_5 and T_6.
5. Close valve F. This is very important, because if the valve is not closed, ensuing operations may damage the entire system. The cell can now be pressurized to the desired confining pressure.

B. Adjusting Confining Pressure

1. Open valves C, C_2, and I (leading to Bourdon gage). Pressure should be zero.
2. Open valve T_1.
3. Slowly open valve C_1 and open the air supply valve K.
4. Adjust the confining pressure regulator to the desired confining pressure as observed on the Bourdon gage. The confining pressure may be measured on the gage by opening valve C, but ***must always be shut*** before opening valve D to measure the pore pressure. This pressure will seal off the sides of the specimen such that all flow measurements result from vertical flow through the sample, and not around it.
5. Close valve C when the desired confining pressure is achieved.

C. Adjusting Pore Pressure

1. Open valves D, D_1, D_2, and the pore pressure inlet valve T_3 to the specimen.
2. Apply a pore pressure to the specimen, adjusting and regulating it with the pressure regulator. The pore pressure must be less than the confining pressure. Keep valve T_2 opened to drain the specimen.

D. Measuring Flow Through the Specimen

1. Watch the tubing within the cell to trace outflow from the specimen. If there is a steady flow of air bubbles through it, the flow is transient. Thus the specimen is

incompletely saturated and more time should be allowed to void the specimen of air so as to achieve steady state flow. From the time that pressure is applied to the specimen, the first observation of transient outflow may occur in as little as a few minutes or in as much as several hours, depending on the permeability of the specimen or how well the specimen was initially de-aired. The time it takes to first observe this will not be known until at least one typical specimen has been tested.

2. Once continuous, steady state is achieved, close valves T_2 and T_3 to the specimen and take an initial burette reading. Take enough timed readings of volume changes in the burette to determine a steady volumetric flow rate through the specimen.
3. If the flow in the burette runs too fast to allow the observer to take timed readings, the pore pressure may be reduced until the volume change is slow enough to read. If the problem still occurs at the lowest regulated pore pressure, say, 6.9 kPa, the regulator should be shut off (valve D_1). Then the pore pressure should be generated and measured as follows:

    a. Determine the difference in height (cm) between the water level in the constant head overflow tank and the bottom of the specimen where pore pressure is applied.
    b. Open valve F leading to the overflow tank, keeping valves D and D_2 open. This will apply a gravity-driven pore pressure to the specimen.
    c. When outflow from the specimen has reached steady state, shut off valve T_3 and take an initial burette reading. Then open the valve again, and measure the volume changes over an appropriate time interval, say, 30 seconds. Repeat this step three times to get an average flow rate. If it becomes necessary to draw up the kerosene-water interface in the burette, do this by shutting off all valves and disconnecting the tubing to the specimen at valve T_3.
    d. Immerse the end of the tubing in a water bottle and, with valves A, D, and D_2 open, apply a backpressure to the burette by turning the handwheel counterclockwise until the water column sufficiently displaces the kerosene.

e. Shut valves A, D, and D_2 and reconnect the tubing at valve T_3. The apparatus is now ready for the next run.

Calculation

For each specimen tested, plot graphs of volume change (cm^3) against time (sec). At steady state, laminar flow, an approximately linear plot should be obtained, whose slope can be found by a linear regression. This slope is the volume flow rate, Q, in cm^3/s. According to Darcy's equation,

$$Q = \frac{khA}{l}$$

where k is the coefficient of permeability, h is the head difference in cm, A is the cross-sectional area, l is the height of the specimen in cm, and h/l or i, is the hydraulic gradient. The head difference, h, across the specimen is the difference between pore pressure at the bottom and pore pressure at the top. Since the pore pressure at the top of the specimen is essentially zero, h is equal to that pressure supplied by the pore pressure regulator.

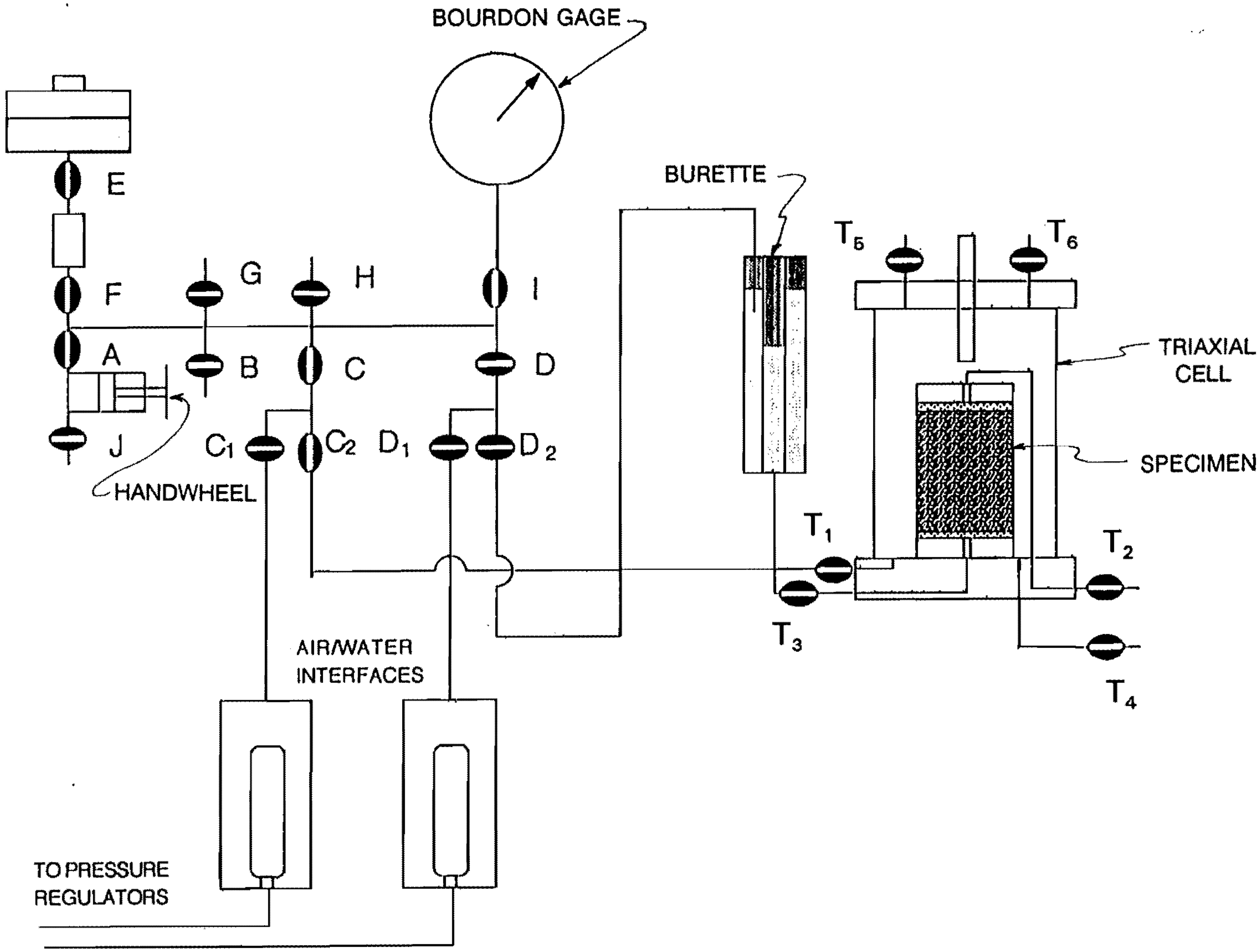

Figure A-2. Constant Head Permeameter Schematic.

METHOD II: FALLING HEAD TEST PROCEDURE (for mixtures with greater than 3% air voids)

Since mixtures with high air void ratios are very permeable, the flow of a fluid through them may be impeded by the narrow orifices in the commonly available constant head apparatus. Thus for highly permeable mixes, a falling head test is more appropriate to measure permeability.

Apparatus

- Plexiglass specimen cylinder with a 100 mm (4 in.) inside diameter. The base of the cylinder should be fitted with a platen of 100 mm (4 in.) outside diameter, and 62.5 mm (2 in.) inside diameter. Bolt the platen into the cylinder wall to form a water-tight seal.
- Top aluminum platen - 62.5 mm (2 in.) inside diameter, 100 mm (4 in.) outside diameter. Inside diameter should be threaded to match lower end of acrylic tubing.
- Acrylic tubing (1.8 m (6 ft) long and 62.5 mm (2 in) diameter), with threads at lower end.
- 2 bronze porous stones, 62.58 mm (2 in.) diameter.
- 62.5 mm (2 in.) PVC coupling fitted with 1/4 - turn valve.
- Flexible rubber tubing.
- Measuring tape.
- Overflow Bucket.
- Stop Watch.

Test Specimen Preparation

The specimen should be of the same size and form as with the constant head test. Saturate the specimen as described earlier.

Mounting the Specimen

Screw one end of the PVC coupling onto the plexiglass tubing. Then screw the plexiglass platen onto the other end of the coupling, and use a sealant to prevent leaks. Clamp the entire assembly vertically to a stand or a wall for support.

Cap the top and bottom of the specimen with the bronze stones, and slip a rubber membrane over the stack. Mount the stack into the specimen cylinder, then make a seal between the top stone and the inside wall of the cylinder with the silicone rubber adhesive. Allow the adhesive to cure for several hours.

When the silicone has cured, place the specimen cylinder on a stand under the clamped apparatus, and fit the platen over the top porous stone. Then make a seal between the platen and the cylinder as with the porous stone. Allow to cure overnight.

Procedure

See diagram of test apparatus (Figure A-3).

1. Supply water by the rubber hose from a water inlet to the 1.8 m (6 ft) plexiglass column. Check to see that there is no leak. Remove all the air bubbles.
2. Open the valve and allow water to flow for some time to saturate the specimen. Close the valve.
3. Measure the head difference, h_1 (cm).
4. Open the valve again and, with a stop watch, record time (t) until the head difference becomes equal to h_2, in cm.
5. Close the flow of water through the specimen by closing the valve.
6. Add more water to the column to make another run. Repeat steps 1-4. Record the temperature of the water.

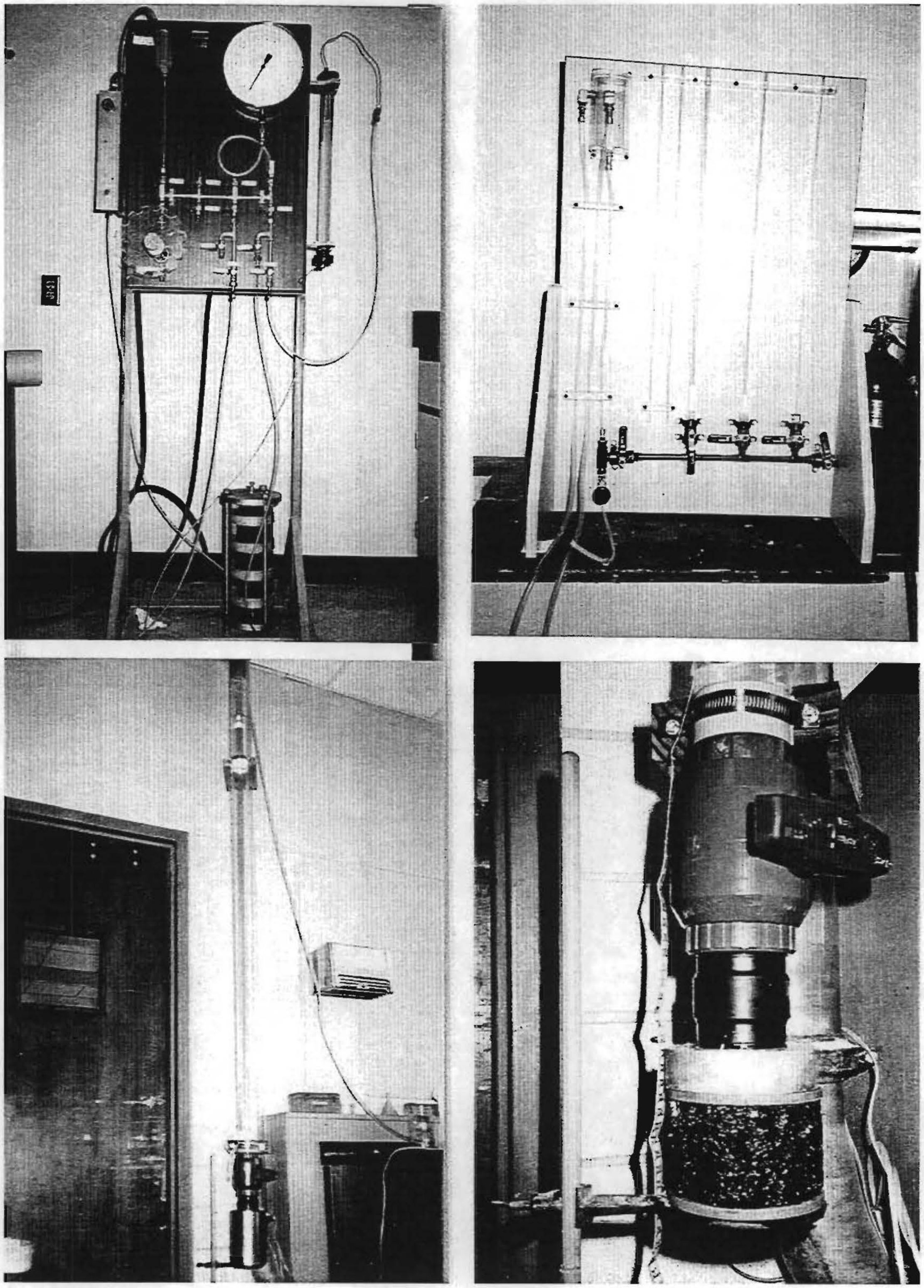

Figure A-3. Falling Head Apparatus.

Calculation

The coefficient of permeability can be expressed by the relation:

$$k = 2.303\frac{al}{At}\log\frac{h_1}{h_2}$$

where a = inside cross-sectional area of tubing, A = cross-sectional area of the specimen, h_1 and h_2 are the initial and final head differences, and t = time in seconds.

PRINCIPLES OF GROUND PENETRATING RADAR

Ground Penetrating Radar operates by transmitting short pulses of electromagnetic energy into the pavement. These pulses (as shown in Figure A-4) are reflected back to the antenna with the amplitude and arrival time that is related to the electrical properties of the pavement layers. The incident wave is reflected at each layer interface and plotted as return voltage against time of arrival in nanoseconds. The reflected energy is collected and displayed as a waveform; Figure A-5 shows a typical example showing amplitudes and arrival times of reflections. Peaks A, B, and C are reflections from the surface, top of the base, and top of the subgrade, respectively. This is a flexible pavement consisting of 17.5 cm (7 in.) of hot mix over a 15-cm (6-in.) granular base over a clay subgrade. The large peak A at 6 nanoseconds is the energy reflected from the surface; peaks B and C represent reflections from the top of the base and subgrade, respectively. The time interval between peaks A and B is the travel time for the radar wave to travel from the surface to the top of the base and back (twice the asphalt thickness). The speed with which the electromagnetic radar wave travels in a particular layer is related to the dielectric constant of that layer. The dielectric constant also determines what percentage of the energy is transmitted and reflected at each layer interface.

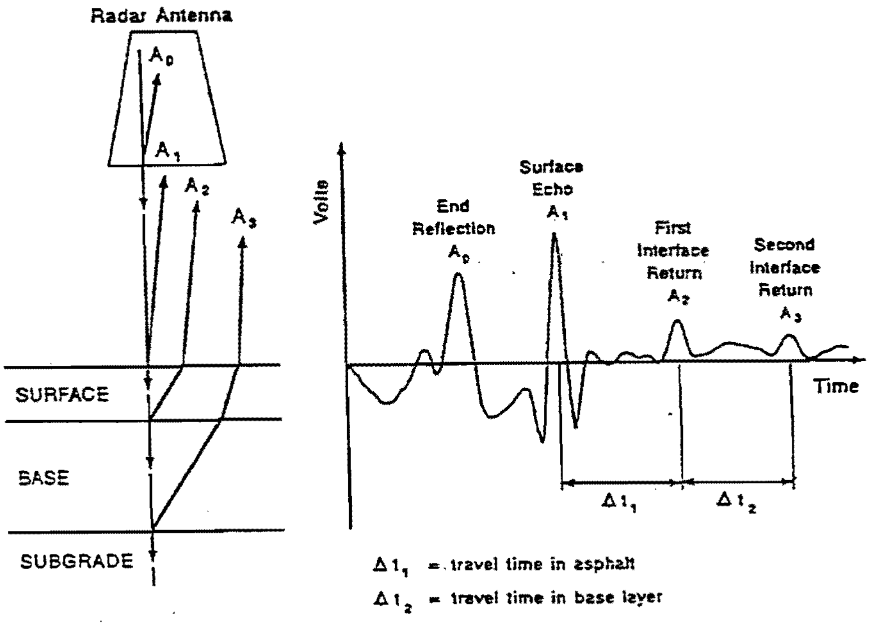

Figure A-4. Principles of Ground Penetrating Radar.

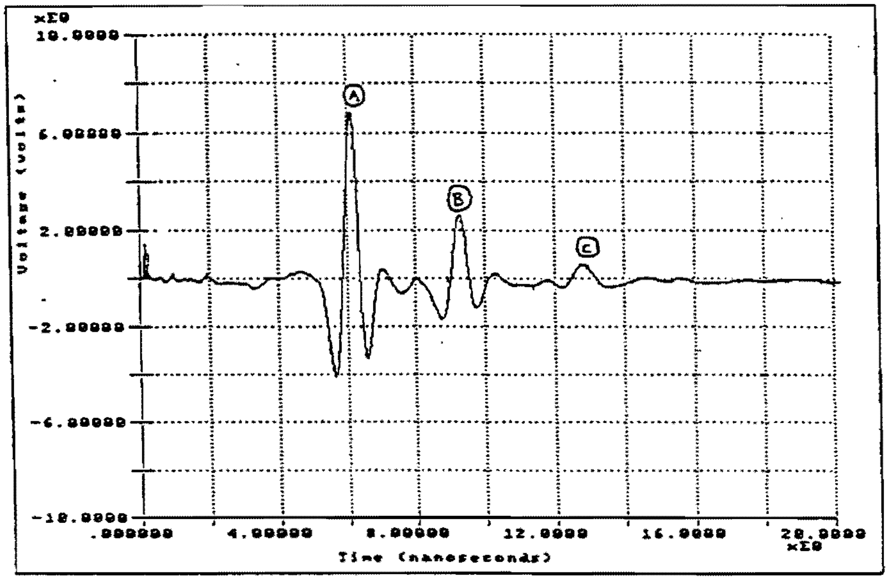

Figure A-5. Typical GPR Waveform.

In pavements, the parameter that most influences the dielectric properties of materials is the moisture content. Table A-1 shows dielectrics for typical pavement materials. As can be seen from this table, the addition of moisture to any of these materials will have a significant influence on the dielectric properties of that layer. For example, a dry limestone base course with 4% by weight of moisture will have a dielectric of around 6; if the moisture content increases to 10%, then the dielectric of the layer would increase to around 11. The impact of a wet base on the trace shown in Figure A-5 would be to increase the amplitude of peak B and to increase the travel time between peaks B and C.

The fact that GPR is sensitive only to changes in dielectric, which mostly equates to changes in layer moisture content, is of major significance. Without these differences in electrical properties, no energy will be reflected at interfaces. Several cases exist in pavements where the layers are so similar electrically that no significant reflections will be detected. Cases like this are common, such as granular base over sand subgrade, or concrete over cement stabilized bases. In these cases, the electrical difference between layers may not be sufficient to permit layer thickness estimates.

Table A-1. Typical Dielectrics for Highway Materials.

Material	Dielectric Constant
Air	1
Water	81
Asphalt	3 - 6
Concrete	6 - 11
Limestone	4 - 8
Clays	5 - 40
Dry Sand	3 - 5
Saturated Sand	20 - 30

The software used in this study automatically measures the amplitudes and time delays of each radar trace received and applies the signal processing described below. Figure A-5 shows a single trace from a section of highway. The user can specify the frequency at which traces are to be collected. In some instances, such as void detection, one trace per 30 cm of pavement may be required. In others, such as layer thickness inventorying, one measurement (one trace) per 30 m may be adequate. In either case, a typical radar survey consists of collecting and processing multiple traces similar to the one shown in Figure A-5.

The principles of GPR applied to highways have been given elsewhere (Maser and Scullion 1991). By automatically monitoring the amplitudes and time delays between peaks, it is possible to calculate layer dielectrics and layer thicknesses, and to estimate the moisture content of granular base material. These equations are summarized below (see Maser and Scullion 1991 for derivations).

$$\varepsilon_a = \left[\frac{1 + A_o/A_m}{1 - A_o/A_m}\right]^2 \tag{A-1}$$

where

ε_a = the dielectric of the asphalt or concrete surfacing layer,

A_o = the amplitude of reflection from the surface in volts (peak A in Figure A-5), and

A_m = the amplitude of reflection from a large metal plate in volts (this represents the 100% reflection case).

$$h_1 = \frac{c \times \Delta t_1}{\sqrt{\varepsilon_a}} \tag{A-2}$$

where

h_1 = the thickness of top layer,

c = a constant obtained from the time calibration procedure, and

Δt_1 = the time delay between peaks A and B of Figure A-5.

$$\sqrt{\varepsilon_b} = \sqrt{\varepsilon_a} \left[\frac{1 - \left[\frac{A_o}{A_m}\right]^2 + \left[\frac{A_1}{A_m}\right]}{1 - \left[\frac{A_o}{A_m}\right]^2 - \left[\frac{A_1}{A_m}\right]} \right] \qquad \text{(A-3)}$$

where

ε_b = the dielectric of the base layer, and

A_1 = the amplitude of reflection from the top of the base layer in volts (peak B in Figure A-5).

$$M = \frac{\sqrt{\varepsilon_b} - 1 - \gamma(\sqrt{\varepsilon_s} - 1)}{\sqrt{\varepsilon_b} - 1 - \gamma(\sqrt{\varepsilon_s} - 22.2)} \qquad \text{(A-4)}$$

where

M = the moisture content of base (% of total wt.),

ε_s = solids dielectric constant (varies from 4 to 8 depending on source material), and

γ = dry density γ_d (lbs/ft^3) divided by density of solids γ_s (~165 lbs/ft^3).

Note that equation A-4 assumes that the density along a highway remains constant. This clearly is not the case and will limit the accuracy of moisture content estimation. However, the moisture content is the major factor which influences measured base dielectric constant ε_b. The relative dielectric constants of air, dry granular base, and water are approximately 1, 6, and 81, respectively. High base dielectrics are almost certainly attributable to high moisture contents. The accuracy of equation A-4 is yet to be determined.

The above equations serve as the basis for analysis of the data collected during this study, as described below. They are based on the assumption that the layer materials are non-conductive and homogenous. This assumption means that the imaginary component of the dielectric constant tends to zero, and the medium does not attenuate the radar signal. Therefore, all of the energy is either reflected or transmitted and none is lost in heating free water in the layer. The assumption of a very low imaginary dielectric from laboratory tests at the Texas Transportation Institute appears to be reasonable for asphalt concrete hot mix. However, it does not seem to be the case for either concrete or wet base course material. Because of the higher attenuation, it is thought that the accuracy of layer thickness estimates for both concrete layers and granular base layers may be less than for hot mix layers. The layer thickness estimates for hot mix asphalts were found to be very good (Maser, Scullion, and Briggs 1991). The accuracy on granular base courses was reasonable, but this was also tied to the inability to physically measure the thickness of existing bases given the intrusion of subgrade materials. The accuracy of these equations for measuring concrete thicknesses is the subject of current research efforts.

APPENDIX B - CONSTRUCTION MONITORING DATA

SAN ANTONIO PAVEMENTS

In February 1992, H.B. Zachry was awarded the contract to overlay 14.75 km (9.167 miles) on IH-10 from the Kendall County line to 4 km (2.5 miles) north of Loop 1604 in San Antonio. Two test sections were constructed: a control section and an experimental section containing rubber-modified asphalt cement. The pavement depth was approximately 1.75 mm. By 1993, the new overlay failed, showing signs of alligator cracking, rutting, and pumping of fine material. Right wheel path rut depths at four locations averaged between 0.8 - 14.7 mm (0.03 - 0.58 in.) with the deepest rut measurement being 27.7 mm (1.09 in.). Following this failure, the outer lane of the overlay was milled, and the RAP material was taken to Colorado Materials Company, where it was added to new aggregate and asphalt to form a recycled CRM asphaltic concrete mix. The Roto-Mill was 4 m (13 ft.) wide x 64 mm (2.5 in.) deep.

On the original IH-10 job, a small size aggregate was used at the start of the job, but by the end of the job, a larger limestone aggregate was in use, the gradation had been changed from dense to gap-graded, and the binder content was increased to give the desired film thickness. Because the coarser material gave better compaction and performance than the dense graded material at the beginning of the original IH-10 job, the recycled mix design used a coarse, gap-graded material.

Prior to laying down the recycled CRM mix as an inlay, an underseal was placed using a tack coat with precoated aggregate over the milled existing pavement. This underseal was later found to be unfavorable since it collected dust and did not particularly enhance the milled surface texture, possibly reducing the potential for bonding with the surface course. The inlay technique was potentially problematic since it had no outlet for stormwater runoff. It was therefore important to maintain near zero permeability to prevent excess water from ponding under the inlay and subsequently leading to stripping and freeze-thaw destruction of the layers below.

CRM Mix Design

Control Mix (Original IH-10):

340 Type D asphaltic concrete with AC-20.

Experimental Mix (Original IH-10):

340 Type D asphaltic concrete containing a rubber-modified asphalt cement, consisting of 82 % Fina AC-10 and 18% Tiregator Type II granulated vulcanized rubber additive.

Final CRM Mix (Recycled IH-10):

30% CRM RAP, 5.7% asphalt, 79.6% aggregate (by weight) retained on the 2 mm sieve. The following materials were incorporated in the CRM RAP Project:

1) Roto Mill - Item 3822 Plain Asphaltic Concrete Pavement (5 cm to 7.62 cm).
2) Seal Coat - Item 316 Aggr. (TY PF GR4).
Item 316 Asph (AC, HFRS-2P, CRS-2P or MC-2400 LTX.
3) Asphalt - Item 3834 Asph. Concrete (Recycled Rubber) (Surf).

Table B-1 presents the laboratory results generated by the Materials and Test Division verification of the mix design.

Table B-1. Verification of CRM Mix Design by TxDOT Division of Materials and Tests (IH-10--Bexar County, June 1992).

Asphalt Rubber Binder (%)	Avg. Ga	Gr	Ge	Avg. Density	VMA	Hveem Stability	Perm. Strain (cm/cm)
6.5	2.242	2.369	2.596	94.6	20.9		1.3×10^{-3}
7.0	2.292	2.350	2.592	97.5	19.6	48	0.5×10^{-3}
7.5	2.263	2.345	2.606	96.5	21.0	38	0.3×10^{-3}
8.0	2.257	2.342	2.623	96.4	21.6	32	0.6×10^{-3}
8.5	2.231	2.301	2.587	96.9	23.0		1.0×10^{-3}

Gradation

International Surfacing, Inc. (ISI) submitted a preliminary mix design to TxDOT. The design specified a maximum coarse aggregate +2 mm content of 74% and 5.5% passing the 0.075 mm sieve, to allow a gap-graded mix. The gradations of mixture components are shown below in Table B-2.

Table B-2. Gradation Percent Retained for Coarse Matrix, High Binder (CMHB) Material (Colorado Materials).

	Percent Retained					
Sieve Size (mm)	Grade 3 Sand Stone	Grade 3 Limestone	Grade 4 Limestone	Grade 10 Limestone	Mfd. Sand	Rubber RAP
22.4	0.0	0.0	0.0	0.0	0.0	0.0
16.0	4.3	0.0	0.0	0.0	0.0	0.0
9.5	95.0	94.3	10.2	0.0	0.0	3.1
4.75	99.5	98.1	96.0	17.5	0.0	28.5
2.00	99.5	99.1	99.0	94.3	18.9	69.9
0.425	99.5	99.3	99.0	98.6	76.9	78.1
0.180	99.5	99.4	99.1	98.9	91.4	86.6
0.075	99.6	99.5	99.2	99.2	96.4	92.4

Chronological Summary of Field Changes

The lab molded density obtained from the above-mentioned design was 97.5%. Table B-3 provides a summary of design and daily mix data. Construction data is presented in Table B-4.

Construction

Equipment and Procedures

Details regarding construction equipment and procedures are shown in Table B-5 and Figure B-1.

Table B-3. Daily Monitoring of Mix Properties for Recycled IH-10 Construction.

Date	Ga	Gr	% 2 mm+	Hveem Stab	% Air Voids	Total Strain	Creep Comp. (1/kPa)	Creep Stiffness (kPa)	Permanent Strain (x 1000)	Recovery Efficiency (%)	Slope of SS Curve (1/s x10^8)	Y Intercept (x 1000)
10-4-93	2.374	2.414	79.2	38	1.657001	1.5	2.16x10-5	46229.3	0.35	75.8	3	1.4
10-5-93	2.368	2.406	81.1	36	1.579385	1.3	1.84x10-5	54489.3	0.33	73.8	2.4	1.2
10-5-93	2.372	2.418	77.1	27	1.902399	1.2	1.78x10-5	56254.3	0.43	64.9	3.8	1.1
10-6-93	2.353	2.416	80.2	35	2.607616	1.7	2.45x10-5	40837.6	0.63	63.5	5.4	1.5
10-7-93	2.354	2.397	77.8	35	1.793909	2.2	2.98x10-5	33570.6	0.66	69.7	3.8	2.1
10-8-93	2.365	2.43	81.1	41	2.674897	1.4	2.01x10-5	49690.5	0.38	73	2.9	1.3
10-11-93	2.365	2.394	79	34	1.211362	2	2.18x10-5	35549.4	0.53	73.5	4.1	1.9
10-12-93	2.379	2.395	77.7	35	0.668058	1.5	2.14x10-5	46836.1	0.38	74.6	3.4	1.4
10-12-93	2.378	2.403	78.9	43	1.040366	1.5	2.10x10-5	47553.1	0.4	71.8	3.7	1.3
10-13-93	2.343	2.411	80.4	38	2.820406	1.7	2.54x10-5	39327.7	0.51	70.8	4.9	1.6
10-14-93	2.355	2.414	82	40	2.444076	1.5	2.16x10-5	46367.2	0.35	76.7	3.3	1.4
10-14-93	2.367	2.413	81	39	1.906341	1.3	1.96x10-4	50952.3	0.42	69.1	3.6	1.2
10-15-93	2.359	2.403	78.4	39	1.831045	1.4	2.03x10-5	49304.4	0.38	71.6	3.2	1.2
10-15-93	2.362	2.412	80.6	40	2.072968	1.8	2.53x10-5	39548.3	0.43	75.2	2.9	1.7

Table B-4. Daily Construction Data at End of Original IH-10 Job.

Date (1993)	Sieve Size				%Asphalt-Rubber	Density (%)	Rice	Ga	VMA (%)	Avg.Core% Air Voids	Stiffness (kPa)	Strain	Slope (1/sec)
	9.5-4.75 mm	4.75-2 mm	+2 mm	-.075 mm									
7/15	37.7	32.3	70.0	6.1	6.8	97.2	2.370	2.304	19.0	11.4	-	-	-
7/15	-	-	-	-	7.3	-	-	-	-	13.0	-	-	-
7/16	39.2	31.9	71.1	6.6	7.9	98.5	2.339	2.304	20.0	13.0	-	-	-
7/17	39.7	33.4	73.1	6.5	7.7	98.2	2.338	2.296	20.1	-	-	-	-
7/17	39.7	33.4	73.1	6.5	7.2	96.7	2.356	2.278	20.3	-	19160.5	1.80	17.0
7/20	43.7	33.7	77.4	6.2	7.7	98.2	2.346	2.304	19.8	6.8	-	-	-
7/23	46.6	30.9	77.5	5.9	7.6	98.5	2.349	2.314	19.3	8.4	44671.1	0.40	2.2
7/24	42.0	34.9	76.9	4.9	7.6	98.1	2.356	2.311	19.4	12.0	45098.6	0.30	2.6
7/25	47.6	28.1	76.2	5.6	7.7	98.1	2.357	2.312	19.5	9.3	52179.5	0.40	2.1
7/27	45.0	32.1	77.1	5.1	8.1	97.7	2.349	2.295	20.4	9.3	44926.2	0.40	2.4
7/28	44.4	33.2	77.8	3.9	7.3	98.9	2.355	2.329	18.6	8.7	49711.2	0.35	3.2
7/31	46.2	29.7	75.9	5.3	8.1	98.4	2.352	2.314	19.8	-	48904.5	0.32	2.4
7/31	49.5	29.4	78.9	5.0	7.7	96.9	2.383	2.309	19.6	-	44402.2	0.40	3.3
8/3	50.2	30.9	81.1	4.3	8.4	97.2	2.356	2.290	20.9	-	47546.2	0.46	2.5
8/6	47.8	32.3	80.1	6.3	8.7	98.7	2.331	2.301	20.8	5.3	34411.7	0.50	3.1
8/6	44.6	33.9	78.5	6.7	9.6	98.8	2.312	2.284	21.2	6.0	37652.3	0.60	3.5
8/6	-	-	-	-	8.6	-	-	-	-	8.8	-	-	-
8/7	46.6	33.8	80.4	4.2	8.2	97.9	2.324	2.275	22.0	6.4	29316.5	0.80	8.1

Table B-5. Weights of Vibratory Roller

Two Wheel Vibratory Roller Caterpillar CB-614		
Total Weight:		11,340 kg
	Front Wheel	5,466 kg
	Rear Wheel	5,874 kg
Dynamic Force per Drum:		
	High Amplitude	9,525 kg
	Low Amplitude	5,670 kg
Total Applied Force per Drum:		
	High Amplitude	15,195 kg
	Low Amplitude	11,567 kg
kg/linear cm at Front Drum:		
	Static	134 kg/cm
	Dynamic	233 kg/cm
	Total Applied	367 kg/cm
kg/linear cm at Rear Drum:		
	Static	144 kg/cm
	Dynamic	233 kg/cm
	Total Applied	377 kg/cm

Three Wheel Roller Ingram 12 ton EB		
Total Unballasted Weight:		8,834 kg
	Rear Wheel	
	Steer Wheel	5,985 kg
Compression:		
	Rear Wheel	2,275 kPa
	Steer Wheel	986 kPa
Total Ballasted Weight:		11,754 kg
	Rear Wheel	8,014 kg
	Steer Wheel	3,741 kg
Compression:		
	Rear Wheel	3,047 kPa
	Steer Wheel	1,289 kPa

Breakdown Roller:
DD-10
19.8-cm width 13.7-cm Diameter
Amplitude 3
Frequency 2500 VPM
Force 83716 N
Weight 11612 kg

Smaller Roller:
DD-90
16.8-cm Width 12.2-cm Diameter
Amplitude 3
Frequency 1850-2600 VPM
Force 115742 N
Weight 9108 kg

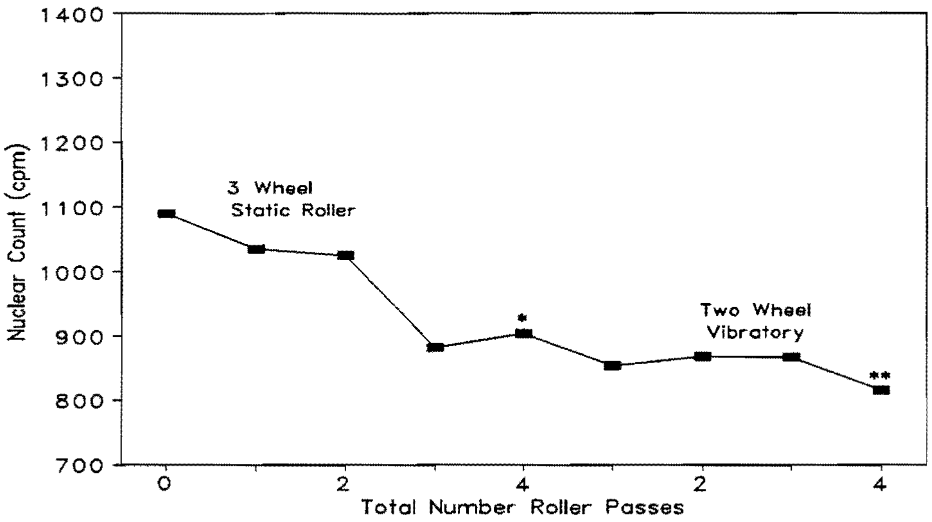

Figure B-1. Rolling Pattern Determinations for Recycled CRM Taken on October 6, 1993.

Problems Encountered and Solutions (Original CRM Job from Materials and Test Division Records)

1) Difficulty with uniform compaction: Because the old existing pavement surface had wheel path rutting depths of 1.87 to 2.5 cm (0.75 to 1.0 in), uniform compaction was very difficult to achieve. Personnel from Redland Stone Products recommended a flush coat of emulsified asphalt be applied to the surface of the pavement in areas where inadequate compaction yielded high air voids, to reduce excessively high water and air permeabilities.

2) Insufficient film thickness: The preliminary mix design offered by ISI produced a somewhat tender and dry mix with low film thickness, causing placement and compaction to be difficult. This was remedied by increasing binder contents from a low of 7.2% to as much as 8.5% towards the end of the job.

3) Shoving of mix under the roller: This was found to be caused by a low coarse aggregate fraction (70% +2 mm material). The problem was alleviated by cutting the discharge temperature of the paver from 149° to 138°C (300° to 280°F), and increasing the +2 mm fraction from 70% to 74%.

4) Substitution of Coastal AC-10 for Fina AC-10: At station number 392+94, Coastal AC-10 replaced Fina AC-10 because Fina was unable to produce any more AC-10 which would meet specifications.

5) Low road densities: This problem was addressed by varying mix temperature, asphalt content, and gradation, as well as continuously reminding the contractor about the rollers keeping up with the laydown machine.

ENVIRONMENTAL MONITORING

On June 17, 1992, a pretest meeting was held in Austin to discuss an air pollutant emissions test program to be conducted by Southwestern Labs, Inc.. Representatives of TxDOT, the Texas Air Control Board, Redland Stone, the H.B. Zachry Company, and International Surfacing, Inc. attended the meeting. It was determined to test several parameters of the emissions under three operating conditions in the baghouse stack of Redland Stone Products Company, producer of the mix. Results are shown in Tables B-6 and B-7.

Table B-6. Summary of Findings from Stack Monitoring.

			Emission Rate (kg/kg mix)								
Temp	Percent Rubber	Plant	VOC	Benzene	Styrene	Naphthalene	2-Methyl-naphthalene	Phenanthrene	Butadiene	Particulates	Opacity (% / 907 kg)
H	0	D	9.0196e-05	6.8303e-07	1.5774e-10	5.1425e-07	8.8337e-07	1.8929e-08	4.8900e-07	2.3356e-05	4.6604e-02
H	0	D	1.0831e-04	2.7806e-03	1.2323e-06	4.0130e-07	6.4303e-07	4.8978e-08	4.1868e-07	3.0987e-05	6.1832e-02
H	0	D	8.3103e-05	2.3131e-03	1.2386e-07	3.5252e-07	5.7165e-07	4.9226e-08	4.7956e-07	2.6212e-05	5.2304e-02
L	18	D	1.1401e-04	3.5029e-03	1.2517e-07	2.5653e-07	4.9296e-07		4.6824e-07	2.3548e-05	4.6987e-02
L	18	D	1.1189e-04	2.7813e-03	6.9164e-08	2.5110e-07	5.7889e-07		3.8643e-07	2.7984e-05	5.5839e-02
L	18	D	1.1461e-04		2.7912e-07	4.9989e-07	6.1185e-07	9.4618e-09	6.9228e-07	2.7707e-05	5.5286e-02
H	18	D	1.1636e-04			3.4395e-07	1.0617e-06	7.6957e-08	5.2141e-07	8.2080e-05	1.6378e-01
H	18	D	5.1968e-05	9.9153e-04	4.8747e-08	1.8555e-07	5.9912e-07		4.3558e-07	1.8266e-05	3.6448e-02
H	18	D	8.6278e-05	2.7768e-03	1.0633e-07	2.6969e-07	6.0409e-07		3.9297e-07	3.4544e-05	6.8930e-02
H	0	R	2.7146e-05	2.0361e-04	-1.6422e+01	2.4314e-07	1.2183e-06		1.4718e-06	3.4027e-05	5.1097e-02
H	0	R	3.4492e-05	2.2194e-04	9.0533e-09	4.0222e-07	1.8365e-06	1.8106e-07	7.6951e-07	5.1370e-05	4.2581e-02
H	0	R	3.1544e-05	1.5665e-04	8.0184e-09	4.0868e-07	1.8106e-06	1.7977e-07	7.6563e-07	7.4313e-05	5.2129e-02
L	18	R	3.4363e-05	1.6361e-04	2.5866e-09	3.4919e-07	1.7330e-06		5.3413e-07	1.8261e-05	4.8516e-02
L	18	R	6.7639e-06			2.7289e-07	1.4226e-06		8.6134e-07	9.8549e-06	5.4968e-02
L	18	R	1.2429e-04			2.9617e-07	1.5908e-06		9.4023e-07	1.0799e-05	5.0065e-02
H	18	R	7.3330e-06	1.2800e-04	2.2762e-08	4.5524e-07	2.0951e-06	2.3280e-07	1.8986e-06	1.3528e-05	5.9613e-02
H	18	R	1.9086e-04	1.5303e-04	2.8116e-07	3.8540e-07	1.7847e-06	2.0046e-07	5.1732e-07	1.3256e-05	6.1419e-02
H	18	R	1.2372e-04	1.0245e-04	2.6125e-08	3.0651e-07	1.6554e-06	2.0305e-07	1.4912e-06	1.1860e-05	7.8452e-02
H	0	C	1.1744e-05	1.8187e-04	1.9915e-08	5.2416e-09	1.3473e-08	7.7228e-10	1.2981e-07	1.1347e-07	9.4340e-03
H	0	C	2.0210e-06	6.6408e-05	7.5604e-09	1.0282e-08	2.9666e-09	2.3617e-09	1.1809e-07	6.2439e-07	1.2459e-02
H	0	C	2.0161e-06	1.4432e-04	2.5163e-08	1.0603e-08	2.5673e-09		1.4771e-07	3.6482e-07	1.2579e-02
H	18	C	4.6316e-06	3.3172e-04	3.0428e-08	1.5915e-08		1.7760e-09	1.7603e-07	2.9309e-06	2.1661e-03
H	18	C	3.6775e-06	2.2340e-04	2.1299e-08				1.9621e-07	1.2619e-06	2.1583e-03
	18	C	4.2405e-06	3.5306e-04	1.9631e-08	1.7477e-08			1.6599e-07	1.1840e-06	4.3956e-03

H = High Temperature (163°C)
L = Low Temperature (149°C)

D = Duininck Bros;

R = Redland Stone;

C = Colorado Materials

Table B-7. Statistical Analysis of Air Emissions Data.

	Conventional HMA (0%)					Modified HMA (18%)					
				Mean	Standard Error				Mean	Standard Dev.	Modified: Conventional HMA
VOC	0.009242	0.007338	0.008462	0.008347	0.000781	0.023434	0.004033	0.004023	0.010497	0.009148	1.2575
Benzene	0.000332	0.000223	0.000353	0.000303	0.000057	0.000182	0.000066	0.000144	0.000131	0.000048	0.432291
Styrene	0.000061	0.000043	0.000039	0.000047	0.000009	0.00004	0.000015	0.00005	0.000035	0.000015	0.737662
Naphthalene	0.000032		0.000035	0.000033	0.000002	0.00001	0.000021	0.000021	0.000017	0.000005	0.521635
2-Methyl-naphthalene						0.000027	0.000006	0.000005	0.000013	0.00001	
Phenanthrene						0.000002	0.000005	0.000004	0.000003	0.000001	
Butadiene	0.000351	0.000392	0.000331	0.000358	0.000025	0.000259	0.000236	0.000295	0.000263	0.000024	0.735021
Particulates	0.005848	0.002518	0.002363	0.003576	0.001608	0.000226	0.001246	0.000728	0.000733	0.000416	0.205079

PERFORMANCE

Rut depth measurements taken prior to recycling are shown below in Table B-8.

Table B-8. Rut Depths Measured in Wheel Path of Original IH-10 CRM Overlay (8 Oct. 1993, Prior to CRM RAP Recycle).

Location	Distance		LRUT (mm)	RRUT (mm)
	(m)	(ft)		
TP4	0	0	0	1.5875
TP4	3.05	10	0	6.35
TP4	6.10	20	1.5875	12.7
TP4	9.15	30	6.35	14.2875
TP4	12.20	40	6.35	19.5
TP4	15.25	50	0	6.35
TP4	18.30	60	0	6.35
TP4	21.35	70	0	11.1125
TP4	24.40	80	0	12.7
TP4	27.45	90	0	12.7
TP4	30.50	100	1.5875	9.525
TP4	33.55	110	0	15.875
TP4	36.60	120	1.5875	6.35
TP4	39.65	130	0	9.525
TP4	42.70	140	1.5875	9.525
TP4	45.75	150	1.5875	17.4625
TP5	0	0	0	7.9375
TP5	3.05	10	0	7.9375
TP5	6.10	20	0	7.9375
TP5	9.15	30	0	7.9375
TP5	12.20	40	0	7.9375
TP5	15.25	50	0	7.9375
TP5	18.30	60	0	7.9375
TP5	21.35	70	0	7.9375
TP5	24.40	80	0	7.9375
TP5	27.45	90	0	7.9375
TP5	30.50	100	0	7.9375
TP5	33.55	110	0	7.9375
TP5	36.60	120	0	7.9375
TP5	39.65	130	0	7.9375
TP5	42.70	140	0	7.9375
TP5	45.75	150	0	7.9375

Table B-8. (Cont.)

Location	Distance		LRUT (mm)	RRUT (mm)
TP6	0	0	3.175	3.96875
TP6	3.05	10	3.175	2.38125
TP6	6.10	20	3.96875	0
TP6	9.15	30	4.7625	3.96875
TP6	12.20	40	3.96875	3.175
TP6	15.25	50	2.38125	2.38125
TP6	18.30	60	3.175	3.96875
TP6	21.35	70	3.175	3.175
TP6	24.40	80	2.38125	2.38125
TP6	27.45	90	3.175	0.79375
TP6	30.50	100	3.175	0.79375
TP6	33.55	110	0.79375	3.175
TP6	36.60	120	0.79375	1.5875
TP6	39.65	130	1.5875	1.5875
TP6	42.70	140	2.38125	3.175
TP6	45.75	150	3.175	3.175
TP7	0	0	1.5875	3.175
TP7	3.05	10	17.4625	3.175
TP7	6.10	20	9.525	14.3875
TP7	9.15	30	10.31875	27.78125
TP7	12.2	40	15.08125	19.05
TP7	15.25	50	26.19375	11.1125
TP7	18.3	60	7.9375	5.55625
TP7	21.35	70	7.14375	7.14375
TP7	24.40	80	16.66875	12.7
TP7	27.45	90	17.4625	14.2875
TP7	30.50	100	11.90625	19.05
TP7	33.55	110	18.25625	19.05
TP7	36.6	120	18.25625	22.225
TP7	39.65	130	16.66875	14.2875
TP7	42.70	140	3.96875	23.8125
TP7	45.75	150	11.1125	17.4625

LABORATORY STUDIES

Rheological data for CRM and control binders are shown in Tables B-9a through B-9e.

Binder

Table B-9a. Binder Viscosity Data, 25°C, 0.1 Rad/sec, 5% Strain.

Viscosity Data
Conditions: 25 Celsius, 0.1 rad/sec, 5% strain
Gap: 500 microns for 0% rubber.
1000 microns for 5% and 10% 40 mesh blends.
1500 microns for 15%, 18%, and 20% 40 mesh blends.
1500 microns for 5% and 10% 10 mesh blends.
2000 microns for 15%, 18%, and 20% 10 mesh blends.
Note: TG = Tire Gator Rubber Type, RS = Rouse Rubber Type

ABM-1

Rubber Type and Mesh Size	Rheological Properties (Units)	Rubber Content				
		0%	5%	10%	15%	18%
TG - 10 mesh	Viscosity (poise)	2413000	4052000	5508000	5961000	6385000
	∂ (degree)	86.7	78.32	64.94	59.13	56.95
	G*/sin(∂) (dyne/cm^2)	241700.802	413767.764	608038.078	694485.52	761756.367
TG - 40 mesh	Viscosity (poise)	2413000	3027000	4184000	5188000	3808000
	∂ (degree)	86.7	77.24	68.31	61.28	55.76
	G*/sin(∂) (dyne/cm^2)	241700.802	310364.916	450281.401	591576.952	460633.551
RS- 40 mesh	Viscosity (poise)	2413000	3464000	3535000	2853000	4586000
	∂ (degree)	86.7	75.47	68.28	61.51	55.18
	G*/sin(∂) (dyne/cm^2)	241700.802	357845.277	380515.425	324610.328	558621.465

ABL-2

Rubber Type and Mesh Size	Rheological Properties (Units)	Rubber Content				
		0%	5%	10%	15%	18%
TG - 10 mesh	Viscosity (poise)	304100	775000	1635000	3688000	4968000
	∂ (degree)	80.21	71.09	57.32	50.3	44.89
	G*/sin(∂) (dyne/cm^2)	30859.3933	81921.4246	194249.997	479335.119	703934.518
TG - 40 mesh	Viscosity (poise)	304100	730500	1699000	3063000	3825000
	∂ (degree)	80.21	70.92	60.74	51.95	46.05
	G*/sin(∂) (dyne/cm^2)	30859.3933	77296.456	194747.821	388965.857	531290.394
RS- 40 mesh	Viscosity (poise)	304100	634600	1242000	2382000	3463000
	∂ (degree)	80.21	74.42	64.48	53.58	46.81
	G*/sin(∂) (dyne/cm^2)	30859.3933	65880.7155	137627.71	296016.035	474977.205

DIAMOND SHAMROCK AC-5

Rubber Type and Mesh Size	Rheological Properties (Units)	Rubber Content				
		0%	5%	10%	15%	20%
TG - 10 mesh	Viscosity (poise)	436100	561000	813000	936000	1305000
	∂ (degree)	81.98	76.59	71.46	65.92	59.71
	G*/sin(∂) (dyne/cm^2)	44040.7512	57672.4299	85750.3064	102521.833	151132.075
TG - 40 mesh	Viscosity (poise)	436100	716000	861000	907000	908000
	∂ (degree)	81.98	74.32	69.02	65.12	59.37
	G*/sin(∂) (dyne/cm^2)	44040.7512	74367.5243	92213.2695	99978.9984	105523.121

MADE AC-5

Rubber Type and Mesh Size	Rheological Properties (Units)	Rubber Content				
		0%	5%	10%	15%	20%
TG - 10 mesh	Viscosity (poise)	572900	1300000	1517000	1598000	1658000
	∂ (degree)	89.01	72.85	69	59.99	53.42
	G*/sin(∂) (dyne/cm^2)	57298.5545	136049.334	162492.759	184539.839	206469.268
TG - 40 mesh	Viscosity (poise)	572900	830000	920000	916000	856000
	∂ (degree)	89.01	79	70.97	64.01	59.19
	G*/sin(∂) (dyne/cm^2)	57298.5545	84553.5048	97318.6967	101905.711	99665.832

YBF, AC-20

Rubber Type and Mesh Size	Rheological Properties (Units)	Rubber Content				
		0%	5%	10%	15%	20%
TG - 40 mesh	Viscosity (poise)	1824000	7641000	7605000	6230000	5159000
	∂ (degree)	84.87	68.56	62.76	57.78	51.99
	G*/sin(∂) (dyne/cm^2)	183133.583	820906.317	855362.605	736400.83	654776.185

Table B-9b. Binder Viscosity Data, 25°C, 0.9796 Rad/sec, 5% Strain.

Viscosity Data

Conditions: 25 Celsius, 0.9796 rad/sec, 5% strain

Gap:
500 microns for 0% rubber.
1000 microns for 5% and 10% 40 mesh blends.
1500 microns for 15%, 18%, and 20% 40 mesh blends.
1500 microns for 5% and 10% 10 mesh blends.
2000 microns for 15%, 18%, and 20% 10 mesh blends.

Note: TG = Tire Gator Rubber Type, RS = Rouse Rubber Type

ABM-1

Rubber Type and Mesh Size	Rheological Properties (Units)	Rubber Content				
		0%	5%	10%	15%	18%
TG - 10 mesh	Viscosity (poise)	2395000	2945000	3004000	2676000	2832000
	∂ (degree)	77.48	73.89	62.33	58.07	57.72
	G*/sin(∂) (dyne/cm^2)	2403291.93	3002842.31	3322715.93	3088755.52	3281369.98
TG - 40 mesh	Viscosity (poise)	2395000	2171000	2321000	2374000	1623000
	∂ (degree)	77.48	69.93	61.81	57.37	56.94
	G*/sin(∂) (dyne/cm^2)	2403291.93	2264208.59	2579633.59	2761403.47	1897019.59
RS- 40 mesh	Viscosity (poise)	2395000	2377000	2016000	1393000	1887000
	∂ (degree)	77.48	75.47	63.48	62.14	55.34
	G*/sin(∂) (dyne/cm^2)	2403291.93	2405444.63	2207109.81	1543485.66	2247311.23

ABL-2

Rubber Type and Mesh Size	Rheological Properties (Units)	Rubber Content				
		0%	5%	10%	15%	18%
TG - 10 mesh	Viscosity (poise)	216500	420000	648000	1215000	1484000
	∂ (degree)	74.28	64.2	54.28	47.67	44.02
	G*/sin(∂) (dyne/cm^2)	220324.201	456985.016	781865.959	1609967.82	2091966.7
TG - 40 mesh	Viscosity (poise)	216500	394000	706000	1042000	1183000
	∂ (degree)	74.28	63.83	54.43	47.99	45.61
	G*/sin(∂) (dyne/cm^2)	220324.201	430046.94	850250.018	1373762.24	1621712.88
RS- 40 mesh	Viscosity (poise)	216500	369000	549000	827000	1061000
	∂ (degree)	74.28	65.93	56.49	48.65	44.72
	G*/sin(∂) (dyne/cm^2)	220324.201	395896.618	645007.775	1079183.14	1477107.88

DIAMOND SHAMROCK AC-5

Rubber Type and Mesh Size	Rheological Properties (Units)	Rubber Content				
		0%	5%	10%	15%	20%
TG - 10 mesh	Viscosity (poise)	324300	341000	423000	420000	503000
	∂ (degree)	76.89	71.3	66.36	60.43	55
	G*/sin(∂) (dyne/cm^2)	326185.946	352660.573	452329.072	473044.647	601523.347
TG - 40 mesh	Viscosity (poise)	324300	413000	434000	412000	363000
	∂ (degree)	76.89	70.12	65.07	61.33	57.64
	G*/sin(∂) (dyne/cm^2)	326185.946	430212.664	468830.615	459991.515	420971.037

MADE AC-5

Rubber Type and Mesh Size	Rheological Properties (Units)	Rubber Content				
		0%	5%	10%	15%	20%
TG - 10 mesh	Viscosity (poise)	555400	816000	884000	738800	645000
	∂ (degree)	85.23	70.48	68.48	61.39	55.15
	G*/sin(∂) (dyne/cm^2)	545960.813	848098.425	930857.192	824386.969	769928.349
TG - 40 mesh	Viscosity (poise)	555400	622000	561000	472000	386000
	∂ (degree)	85.23	76.55	70.15	64.95	60.2
	G*/sin(∂) (dyne/cm^2)	545960.813	626494.048	584270.341	510378.283	435746.538

YBF, AC-20

Rubber Type and Mesh Size	Rheological Properties (Units)	Rubber Content				
		0%	5%	10%	15%	20%
TG - 40 mesh	Viscosity (poise)	1411000	3969000	3493000	2657000	1925000
	∂ (degree)	75.92	52.88	51.23	51.31	51.42
	G*/sin(∂) (dyne/cm^2)	1425028.12	4876053.15	4388731.98	3334614.98	2412228.47

Table B-9c. Binder Viscosity Data, 60°C, 0.1 Rad/sec, 5% Strain.

Viscosity Data

Conditions: 60 Celsius, 0.1 rad/sec, 5% strain

Gap: 500 microns for 0% rubber.
1000 microns for 5% and 10% 40 mesh blends.
1500 microns for 15%, 18%, and 20% 40 mesh blends.
1500 microns for 5% and 10% 10 mesh blends.
2000 microns for 15%, 18%, and 20% 10 mesh blends.

Note: TG = Tire Gator Rubber Type, RS = Rouse Rubber Type

ABM-1

Rubber Type and Mesh Size	Rheological Properties (Units)	Rubber Content				
		0%	5%	10%	15%	18%
TG - 10 mesh	Viscosity (poise)	2192	5798	35900	37290	60070
	∂ (degree)	89.93	86.28	48.04	59.72	54.55
	G*/sin(∂) (dyne/cm^2)	219.200164	581.024248	4827.791	4318.11545	7373.97674
TG - 40 mesh	Viscosity (poise)	2192	5302	9797	18180	47550
	∂ (degree)	89.93	86.38	83.55	77.35	55.4
	G*/sin(∂) (dyne/cm^2)	219.200164	531.26004	985.94088	1863.22843	5776.68885
RS- 40 mesh	Viscosity (poise)	2192	5880	9776	21850	37530
	∂ (degree)	89.93	88.34	84.08	67.53	64.49
	G*/sin(∂) (dyne/cm^2)	219.200164	588.246893	982.841743	2364.51544	4158.40395

ABL-2

Rubber Type and Mesh Size	Rheological Properties (Units)	Rubber Content				
		0%	5%	10%	15%	18%
TG - 10 mesh	Viscosity (poise)	1221	4162	35930	50890	114200
	∂ (degree)	89.86	85.21	40.51	68.59	61.4
	G*/sin(∂) (dyne/cm^2)	122.100365	417.658738	5531.2638	5466.21385	13007.0923
TG - 40 mesh	Viscosity (poise)	1221	4352	10950	34010	100500
	∂ (degree)	89.86	78.15	84.92	77.03	6.01
	G*/sin(∂) (dyne/cm^2)	122.100365	444.676809	1099.31821	3490.03978	95986.7506
RS- 40 mesh	Viscosity (poise)	1221	3205	7415	22645	63390
	∂ (degree)	89.86	89.41	87.46	80.95	68.39
	G*/sin(∂) (dyne/cm^2)	122.100365	320.516998	742.229263	2293.04552	6818.24158

DIAMOND SHAMROCK AC-5

Rubber Type and Mesh Size	Rheological Properties (Units)	Rubber Content				
		0%	5%	10%	15%	20%
TG - 10 mesh	Viscosity (poise)	723.8	1691	2570	4575	11970
	∂ (degree)	89.89	81.94	83.42	71.5	60.18
	G*/sin(∂) (dyne/cm^2)	72.3801336	170.787101	258.704166	482.430401	1379.68159
TG - 40 mesh	Viscosity (poise)	723.8	2138	3162	5473	10300
	∂ (degree)	89.89	86.68	82.59	72.75	64.27
	G*/sin(∂) (dyne/cm^2)	72.3801336	214.159448	318.862981	573.077228	1143.36533

MADE AC-5

Rubber Type and Mesh Size	Rheological Properties (Units)	Rubber Content				
		0%	5%	10%	15%	20%
TG - 10 mesh	Viscosity (poise)	884.8	12100	19920	22920	47480
	∂ (degree)	90.08	35.3	39.86	53.98	48.02
	G*/sin(∂) (dyne/cm^2)	88.4800861	2093.94293	3108.06401	2833.7883	6357.0612
TG - 40 mesh	Viscosity (poise)	864.8	2798	5916	12650	24810
	∂ (degree)	90.08	83.26	75.87	62.56	51.05
	G*/sin(∂) (dyne/cm^2)	88.4800861	281.747208	610.057879	1425.36261	3190.19922

YBF, AC-20

Rubber Type and Mesh Size	Rheological Properties (Units)	Rubber Content				
		0%	5%	10%	15%	20%
TG - 40 mesh	Viscosity (poise)	2698	16146	30660	44640	70280
	∂ (degree)	89.88	82.74	76.17	67.29	58.09
	G*/sin(∂) (dyne/cm^2)	269.800592	1627.64926	3157.54031	4839.17998	8279.15411

Table B-9d. Binder Viscosity Data, 60°C, 1.0 Rad/sec, 5% Strain.

Viscosity Data

Conditions: 60 Celsius, 1.0 rad/sec, 5% strain

Gap:
500 microns for 0% rubber.
1000 microns for 5% and 10% 40 mesh blends.
1500 microns for 15%, 18%, and 20% 40 mesh blends.
1500 microns for 5% and 10% 10 mesh blends.
2000 microns for 15%, 18%, and 20% 10 mesh blends.

Note: TG = Tire Gator Rubber Type. RS = Rouse Rubber Type

ABM-1

Rubber Type and Mesh Size	Rheological Properties (Units)	Rubber Content				
		0%	5%	10%	15%	18%
TG - 10 mesh	Viscosity (poise)	2209	4925	14620	19150	26470
	∂ (degree)	89.84	85.74	64.85	67.03	62.38
	G*/sin(∂) (dyne/cm^2)	2209.00862	4938.64478	16372.1111	20799.1901	29874.4513
TG - 40 mesh	Viscosity (poise)	2209	4312	7884	13450	21480
	∂ (degree)	89.84	85.49	81.81	77.77	61.38
	G*/sin(∂) (dyne/cm^2)	2209.00862	4325.39347	7965.23796	13762.3381	24469.8375
RS- 40 mesh	Viscosity (poise)	2209	4949	7371	12960	21180
	∂ (degree)	89.84	85.74	81.93	69.03	67.03
	G*/sin(∂) (dyne/cm^2)	2209.00862	4962.71127	7444.72415	13879.2568	23004.0128

ABL-2

Rubber Type and Mesh Size	Rheological Properties (Units)	Rubber Content				
		0%	5%	10%	15%	18%
TG - 10 mesh	Viscosity (poise)	1211	3496	13040	29750	53420
	∂ (degree)	88.6	84.12	59.73	65.28	57.29
	G*/sin(∂) (dyne/cm^2)	1211.36164	3514.49146	15098.5489	32751.2651	63488.2134
TG - 40 mesh	Viscosity (poise)	1211	3293	8692	22610	42720
	∂ (degree)	88.6	83.39	79.65	70.57	55.12
	G*/sin(∂) (dyne/cm^2)	1211.36164	3315.03654	8835.77199	23975.4431	52075.2653
RS- 40 mesh	Viscosity (poise)	1211	2656	6090	17000	35400
	∂ (degree)	88.6	86.98	82.81	73.96	62.58
	G*/sin(∂) (dyne/cm^2)	1211.36164	2659.69395	6138.26884	17688.6413	39880.3919

DIAMOND SHAMROCK AC-5

Rubber Type and Mesh Size	Rheological Properties (Units)	Rubber Content				
		0%	5%	10%	15%	20%
TG - 10 mesh	Viscosity (poise)	726.4	1189	1814	3057	5640
	∂ (degree)	89.35	85.9	84.06	77.81	70.78
	G*/sin(∂) (dyne/cm^2)	726.446758	1192.05082	1823.79252	3127.51764	6184.72957
TG - 40 mesh	Viscosity (poise)	726.4	1604	2341	3580	5894
	∂ (degree)	89.35	85.93	82.27	76.47	69.25
	G*/sin(∂) (dyne/cm^2)	726.446758	1608.05553	2362.46635	3682.19077	6302.83431

MADE AC-5

Rubber Type and Mesh Size	Rheological Properties (Units)	Rubber Content				
		0%	5%	10%	15%	20%
TG - 10 mesh	Viscosity (poise)	884.5	4072	6493	10080	16610
	∂ (degree)	89.95	64.12	60.96	62.3	55.24
	G*/sin(∂) (dyne/cm^2)	884.500338	4525.90388	7426.67746	11384.7723	20217.9552
TG - 40 mesh	Viscosity (poise)	884.5	2112	3878	6373	9814
	∂ (degree)	89.95	83.88	75.7	66.46	59.25
	G*/sin(∂) (dyne/cm^2)	884.500338	2124.10601	4002.00015	6951.49599	11419.5158

YBF, AC-20

Rubber Type and Mesh Size	Rheological Properties (Units)	Rubber Content				
		0%	5%	10%	15%	20%
TG - 40 mesh	Viscosity (poise)	2698	12380	20120	25280	33890
	∂ (degree)	89.61	81.04	74.04	66.74	60.81
	G*/sin(∂) (dyne/cm^2)	2698.06253	12532.9379	20926.6471	27516.4875	38819.8591

Table B-9e. Bending Beam Rheometer Data for Asphalt Rubber Blends.

Bending Beam Rheometer
Asphalt= Diamond Shamrock AC-5
Test Temperature= -15 Celsius

		Percent Rubber:	0%	5%	10%	15%
	Loading Time, s	Load, gms	Creep Stiffness MPa			
Tire Gator-10 mesh	15	100	268	224	193	133
	30	100	220	182	154	107
	60	100	180	144	123	84
	120	100	145	111	95	66
	240	100	116	85	74	51
		m Value=	0.34	0.38	0.38	0.37

		Percent Rubber:	0%	5%	10%	15%
	Loading Time, s	Load, gms	Creep Stiffness MPa			
Tire Gator-40 mesh	15	100	268	166	138	107
	30	100	220	143	108	83
	60	100	180	121	83	64
	120	100	145	100	62	48
	240	100	116	80	46	36
		m Value=	0.34	0.29	0.42	0.44

Aggregate

Aggregate and CRM gradations for the construction projects are shown in Tables B-10a and B-10b.

Table B-10a. Gradations for Original IH-10 Construction.

Sample B: Gradation of Extracted Aggregate				
Sieve Size (mm)	Weight Agg.	Percent Ret.	Accum % Ret.	Percent Passing
12.7	0.0	0.00	0.0	100.0
9.5	22.6	2.55	2.5	97.5
4.75	95.9	10.80	13.4	86.6
2.00	223.2	25.15	38.5	61.5
0.425	259.9	29.28	67.8	32.2
0.180	150.6	16.97	84.7	15.3
0.075	63.9	7.20	91.9	8.1
-0.075	70.6	7.95	99.9	0.1

Sample C				
Sieve Size (mm)	Weight of Agg.	% Ret.	Accum % Ret.	Percent Passing
12.700	0.0	0.00	0.0	100.0
9.500	28.4	3.43	3.4	96.6
4.750	99.8	12.04	15.5	84.5
2.000	206.0	24.86	40.3	59.7
0.425	227.3	27.43	67.8	32.2
0.180	119.6	14.43	82.2	17.8
0.075	57.2	6.90	89.1	10.9
-0.075	89.8	10.84	99.9	0.1

Table B-10a. (Cont.)

Sample B: Gradation of Extracted Rubber From Field Mix				
Sieve	Weight Agg. (g)	Percent Ret.	Accum % Ret.	Percent Passing
9.500	0.0	0.00	0.0	100.0
4.750	0.3	3.02	3.0	97.0
2.000	1.0	10.05	13.1	86.9
0.600	7.5	75.38	88.4	11.6
0.425	0.7	7.04	95.5	4.5
0.180	0.3	3.02	98.5	1.5
0.150	0.0	0.00	98.5	1.5
0.075	0.1	1.01	99.5	0.5
-0.075	0.0	0.00	99.5	0.5

Sample C: Gradation of Extracted Rubber from Field Mix				
Sieve Size (mm)	Weight Agg. (g)	Percent Ret.	Accum % Ret.	Percent Passing
9.500	0.0	0.00	0.0	100.0
4.750	0.1	1.12	1.1	98.9
2.000	1.5	16.85	18.0	82.0
0.600	6.5	73.03	91.0	9.0
0.425	0.5	5.62	96.6	3.4
0.180	0.2	2.25	98.9	1.1
0.150	0.0	0.00	98.9	1.1
0.075	0.1	1.12	100.0	0.0
-0.075	0.0	0.00	100.0	0.0

Table B-10a. (Cont.)

Gradation of New Rubber (never been extracted)				
Sieve Size (mm)	Weight of Aggr. (g)	Percent Retained	Cumulative % Retained	% Passing
9.500	0.0	0.00	0.0	100.0
4.750	0.0	0.00	0.0	100.0
2.000	0.0	0.27	0.3	99.7
0.600	5.5	50.00	50.3	49.7
0.425	2.4	21.82	72.1	27.9
0.180	2.3	20.91	93.0	7.0
0.150	0.1	0.91	93.9	6.1
0.075	0.5	4.55	98.5	1.5
-0.075	0.1	0.91	99.4	0.6

Table B-10b. Gradations for Tyler CRM Job.

DATE 10-15-92 MECHANICAL ANALYSIS OF EXTRACTED AGGREGATE				
SAMPLE NO. 1				
Sieve Size (mm)	Wt. of Aggregate (g)	% Retained	Accum %Retained	% Passing
12.700	0	0.00	0.00	100.00
9.500	65.8	6.98	6.98	93.02
4.750	367.5	38.97	45.95	54.05
2.000	227.6	24.14	70.08	29.92
0.425	128.3	13.61	83.69	16.31
0.180	51.8	5.49	89.18	10.82
0.075	66.9	7.09	96.28	3.72
-0.075	31	3.29	99.57	0.43
DATE 10-15-92				
MECHANICAL ANALYSIS OF EXTRACTED AGGREGATE				
SAMPLE NO. 2				
Sieve Size (mm)	Wt. of Aggregate (g)	% Retained	Accum. % Retained	% Passing
12.700	0	0.00	0.00	100.00
9.500	58.9	6.25	6.25	93.75
4.750	373.1	39.57	45.82	54.18
2.000	229.8	24.37	70.19	29.81
0.425	124.9	13.25	83.43	16.57
0.180	50.3	5.33	88.77	11.23
0.075	68.9	7.31	96.08	3.92
-0.075	35.3	3.74	99.82	0.18

Table B-10b. (Cont.)

DATE 10-15-92				
MECHANICAL ANALYSIS OF EXTRACTED AGGREGATE				
SAMPLE NO. 3				
Sieve Size (mm)	Wt. of Aggregate (g)	% Retained	Cumulative % Retained	% Passing
12.700	0	0.00	0.00	100.00
9.500	72	7.63	7.63	92.37
4.750	395.2	41.90	49.53	50.47
2.000	215.3	22.82	72.35	27.65
0.425	120.2	12.74	85.09	14.91
0.180	47.5	5.04	90.13	9.87
0.075	63.9	6.77	96.90	3.10
-0.075	28.7	3.04	99.95	0.05

DATE 11-01-92				
MECHANICAL ANALYSIS OF EXTRACTED AGGREGATE				
SAMPLE BLENDED FROM RAW MATERIAL				
Sieve Size (mm)	Wt. of Aggregate (g)	% Retained	Cumulative % Retained	% Passing
12.700	0	0.00	0.00	100.00
9.500	49	4.94	4.94	95.06
4.750	353.5	35.60	40.54	59.46
2.000	259.9	26.18	66.71	33.29
0.425	166.6	16.78	83.49	16.51
0.180	66.3	6.68	90.17	9.83
0.075	78.3	7.89	98.06	1.94
-0.075	19.3	1.94	100.00	0.00

Table B-10b. (Cont.)

DATE 11-01-92				
MECHANICAL ANALYSIS OF EXTRACTED RUBBER				
Sieve Size (mm)	Wt. of Aggregate (g)	% Retained	Cumulative % Retained	% Passing
12.700	0	0.00	0.00	100.00
9.500	0	0.00	0.00	100.00
4.750	0	0.00	0.00	100.00
2.000	0.07	0.53	0.53	99.47
0.600	8.862	66.76	67.29	32.71
0.425	1.608	12.11	79.40	20.60
0.180	2.484	18.71	98.12	1.88
PAN	0.25	1.88	100.00	0.00

DATE 11-01-92				
MECHANICAL ANALYSIS OF ORIGINAL RUBBER				
Sieve Size (mm)	Wt. of Aggregate (g)	% Retained	Cumulative % Retained	% Passing
4.750	0	0.00	0.00	100.00
2.000	0	0.00	0.00	100.00
0.600	0.8	5.33	5.33	94.67
0.425	0.965	6.43	11.77	88.23
0.180	10.652	71.01	82.78	17.22
0.150	1.028	6.85	89.63	10.37
0.075	1.023	6.82	96.45	3.55
-0.075	0.48	3.20	99.65	0.35

Mixture

IH-20 Test Section (Tyler)

In September 1992, a test section using an asphalt rubber mix was placed on IH-20 between Tyler and Longview. Mix design information, samples of the ground rubber used on the job, and samples of milled material from the IH-20 site were obtained and taken back to TTI for further study.

Sets of specimens were remolded using the milled rubber material, molded using the raw materials incorporating rubber, and molded without the rubber. The remolded material specimens (MLM) averaged 6.8% air voids; the specimens mixed using virgin materials (RLM) averaged 2.4%, and the specimens mixed with no rubber (CLM) averaged 0% air voids. Indirect tensile strength tests were performed on these specimens. The data provided in Table B-11 shows that the specimens remolded from the milled material had the highest strength and modulus, but the strain at the strength was the lowest of the three mixtures. This is the normal tradeoff (higher strength, lower strain at 'failure') to be expected if the asphalt were hardened with age and reheating/mixing/compacting. The virgin mixture had an intermediate strength, modulus, and strain.

Table B-11. Indirect Tensile Data for Molded Specimens from IH-20.

Sample	Height (cm)	Rice	BSG	Air Voids (%)	Load (kg)	Deform (mm)	Stress (kPa)	Strain
CLM	5.151	2.332	2.330	0.086	376	0.889	437.7	0.0182
CLM	5.210	2.332	2.328	0.172	388	0.6985	448.9	0.0143
CLM	5.093	2.332	2.339	0	427	0.7112	505.3	0.0146
MLM	5.532	2.360	2.202	6.695	821	0.4140	894.3	0.0085
MLM	5.552	2.360	2.178	7.712	800	0.4572	867.4	0.0094
MLM	5.519	2.360	2.218	6.017	864	0.3175	942.9	0.0065
RLM	5.398	2.328	2.269	2.534	596	0.381	665.6	0.0078
RLM	5.403	2.328	2.267	2.62	660	0.635	735.8	0.0130
RLM	5.347	2.328	2.278	2.148	668	0.6604	752.6	0.0135

Results of the Gyratory Test Machine (GTM) data are shown in Tables B-12 and B-13.

Table B-12. Results of Laboratory Testing of GTM Samples at TTI.

Specimen ID	CRM RAP Content (%)	Bulk Specific Gravity	Rice Specific Gravity	Air Void Content (%)	Hveem Stability
SHM0.R1	0	2.402	2.445	1.759	11.3
SHM0.R2	0	2.418	2.445	1.104	11.0
SHM0.R3	0	2.417	2.445	1.145	16.0
SMH0.R1	0	2.409	2.442	1.351	13.4
SMH0.R2	0	2.413	2.442	1.188	23.5
SMH0.R3	0	2.423	2.442	0.778	18.3
SMM0.R1	0	2.416	2.434	0.740	11.7
SMM0.R2	0	2.414	2.434	0.822	12.4
SMM0.R3	0	2.416	2.434	0.740	16.6
SML0.R1	0	2.393	2.438	1.846	19.5
SML0.R2	0	2.401	2.438	1.518	18.4
SML0.R3	0	2.395	2.438	1.764	15.4
SLM0.R1	0	2.402	2.438	1.477	38.5
SLM0.R2	0	2.406	2.438	1.313	43.9
SLM0.R3	0	2.399	2.438	1.600	49.9
SHM3.R1	30	2.388	2.433	1.850	17.2
SHM3.R2	30	2.387	2.433	1.891	21.5
SHM3.R3	30	2.387	2.433	1.891	17.0
SMH3.R1	30	2.380	2.432	2.138	18.0
SMH3.R2	30	2.382	2.432	2.056	19.0
SMH3.R3	30	2.391	2.432	1.686	20.2
SMM3.R1	30	2.362	2.423	2.518	16.1
SMM3.R2	30	2.370	2.423	2.187	17.0
SMM3.R3	30	2.377	2.423	1.898	27.9
SML3.R1	30	2.353	2.427	3.049	16.7
SML3.R2	30	2.354	2.427	3.008	11.2
SML3.R3	30	2.355	2.427	2.967	13.6
SHM10.R1	100	2.358	2.420	2.562	27.7
SHM10.R2	100	2.359	2.420	2.521	42.6
SHM10.R3	100	2.367	2.420	2.190	30.0
SMM10.R1	100	2.339	2.379	1.681	13.2
SMM10.R2	100	2.343	2.379	1.513	8.4
SMM10.R3	100	2.347	2.379	1.345	9.3
SLM10.R1	100	2.352	**	**	11.6
SLM10.R2	100	2.345	**	**	9.6
SLM10.R3	100	2.335	**	**	11.9

Table B-12. (Cont.)

Specimen ID	CRM RAP Content (%)	Bulk Specific Gravity	Rice Specific Gravity	Air Void Content (%)	Hveem Stability
SLM3.R1	30	2.313	**	**	19.0
SLM3.R2	30	2.329	**	**	13.9
SLM3.R3	30	2.321	**	**	17.5
STM3.R1	30	**	2.416	**	**
STM3.R2	30	2.403	2.416	0.538	17.4
STM3.R3	30	2.404	2.416	0.497	22.7
SRM3.R1	30	2.400	2.401	0.042	**
SRM3.R2	30	2.410	2.401	-0.375	17.5
SRM3.R3	30	2.394	2.401	0.292	12.1
STL3.R1	30	2.376	2.409	1.370	16.0
STL3.R2	30	2.391	2.409	0.747	16.5
STL3.R3	30	2.385	2.409	0.996	13.0
SRL3.R1	30	2.406	2.409	0.125	17.8
SRL3.R2	30	2.396	2.409	0.540	14.2
SRL3.R3	30	2.397	2.409	0.498	20.1
PAVE2	0	2.309	2.387	3.268	**
SPEC 1	30	2.275	2.424	6.147	17.3
SPEC 2	30	2.270	2.424	6.353	**
SPEC 3	30	2.216	2.424	8.581	**
SPEC 4	30	2.222	2.424	8.333	13.8
SPEC 5	30	2.181	2.424	10.025	9.9
SPEC 6	30	2.203	2.424	9.117	**
SPEC 1A	30	2.320	2.424	5.000	26.4
SPEC 2A	30	2.320	2.424	4.500	22.3

** Data unavailable

Table B-13. Final Data for Air Voids at Increments of 10 Gyrations (max. 250 Gyrations).

Gyrations	SLM0	SML0	SMM0	SMH0	SHM0	SLM3	SML3	SMM3	SMH3
1	23.586	22.543	17.826	20.327	16.553	20.991	20.764	19.332	16.301
10	16.796	12.667	9.260	10.463	6.842	15.093	11.473	9.510	8.309
20	13.301	9.431	6.425	7.386	4.460	12.124	8.566	6.697	5.721
30	11.302	7.724	4.941	5.789	3.431	10.398	7.027	5.281	4.402
40	9.944	6.614	3.987	4.747	2.968	9.212	6.012	4.479	3.655
50	8.943	5.792	3.370	4.026	2.706	8.351	5.294	3.945	3.183
60	8.165	5.161	2.977	3.523	2.513	7.683	4.767	3.586	2.853
70	7.538	4.683	2.727	3.158	2.356	7.142	4.367	3.313	2.607
80	7.019	4.313	2.543	2.890	2.218	6.694	4.045	3.099	2.416
90	6.587	4.032	2.389	2.679	2.096	6.314	3.789	2.929	2.257
100	6.219	3.805	2.257	2.510	1.985	5.996	3.572	2.790	2.126
110	5.904	3.614	2.139	2.367	1.880	5.727	3.383	2.673	2.009
120	5.630	3.461	2.036	2.242	1.782	5.493	3.226	2.570	1.906
130	5.386	3.325	1.941	2.130	1.683	5.283	3.082	2.473	1.816
140	5.167	3.204	1.851	2.027	1.589	5.102	2.954	2.384	1.730
150	4.966	3.097	1.770	1.934	1.497	4.934	2.842	2.303	1.651
160	4.788	2.998	1.689	1.847	1.408	4.789	2.740	2.217	1.583
170	4.624	2.908	1.612	1.767	1.321	4.610	2.643	2.138	1.517
180	4.475	2.823	1.535	1.691	1.233	4.533	2.553	2.063	1.453
190	4.337	2.740	1.465	1.620	1.148	4.420	2.469	1.990	1.398
200	4.214	2.823	1.399	1.554	1.058	4.316	2.385	1.928	1.339
210	4.100	2.588	1.333	1.488	0.971	4.221	2.303	1.866	1.293
220	3.991	2.518	1.272	1.427	0.881	4.133	2.226	1.809	1.249
230	3.894	2.448	1.210	1.370	0.790	4.051	2.151	1.758	1.200
240	3.796	2.380	1.149	1.313	0.700	3.976	2.078	1.703	1.159
250	3.708	2.317	1.090	1.255	0.610	3.908	2.014	1.641	1.119

Table B-13. (Cont.)

	SHM3	STL3	STM3	SRL3	SRM3	SLM10	SMM10	SHM10
Gyrations								
1	14.679	17.172	17.753	15.989	14.475	15.612	10.814	5.842
10	6.040	8.159	7.950	6.791	5.444	10.264	5.344	2.748
20	3.987	5.539	5.144	4.259	3.550	8.034	3.902	2.128
30	3.212	4.323	3.725	3.206	2.824	6.844	3.256	1.839
40	2.816	3.703	2.961	2.645	2.361	6.094	2.887	1.655
50	2.551	3.297	2.495	2.270	2.010	5.571	2.652	1.513
60	2.362	2.992	2.151	1.979	1.716	5.180	2.488	1.403
70	2.178	2.738	1.871	1.740	1.458	4.875	2.366	1.309
80	2.030	2.517	1.632	1.539	1.228	4.619	2.270	1.230
90	1.896	2.316	1.422	1.361	1.021	4.401	2.191	1.158
100	1.776	2.138	1.235	1.199	0.826	4.212	2.122	1.089
110	1.666	1.970	1.065	1.047	0.659	4.050	2.059	1.026
120	1.558	1.818	0.908	0.908	0.495	3.905	1.994	0.959
130	1.455	1.672	0.758	0.767	0.336	3.777	1.931	0.900
140	1.359	1.535	0.621	0.637	0.189	3.662	1.650	0.840
150	1.266	1.405	0.489	0.506	0.045	3.556	1.816	0.781
160	1.181	1.281	0.363	0.385	-0.098	3.458	1.760	0.725
170	1.093	1.158	0.244	0.270	-0.231	3.363	1.711	0.667
180	1.014	1.041	0.124	0.158	-0.357	3.276	1.668	0.608
190	0.935	0.926	0.010	0.041	-0.483	3.188	1.621	0.545
200	0.858	0.813	-0.103	-0.065	-0.606	3.114	1.129	0.485
210	0.781	0.707	-0.207	-0.174	-0.730	3.042	1.538	0.422
220	0.709	0.599	-0.317	-0.277	-0.849	2.977	1.502	0.337
230	0.641	0.493	-0.419	-0.374	-0.964	2.921	1.464	0.296
240	0.575	0.396	-0.513	-0.474	-1.081	2.865	1.430	0.236
250	0.511	0.290	-0.613	-0.573	-1.193	2.811	1.394	0.168

0 = No CRM RAP; 3 = 30% CRM RAP; 10 = 100% CRM RAP

It was found during the course of this study that, if the laboratory creep test is conducted on short (*e.g.*, 50.8-63.5 mm tall) specimens, end effects are significant (especially if the specimens are capped). This is illustrated in Figure B-2 where it is shown that a 152.4 mm tall specimen enters tertiary creep at the same load level applied to a 50.8 mm tall specimen which does not enter tertiary creep. The shorter specimens are more convenient from a compaction and coring standpoint. However, the end effects demonstrated here suggest that the results of the test should be treated as test properties, not material properties, and that specimen preparation must be consistent and precise. The role of aggregate size versus height of the specimen has not been studied here, but its effect on creep results should be studied in the future.

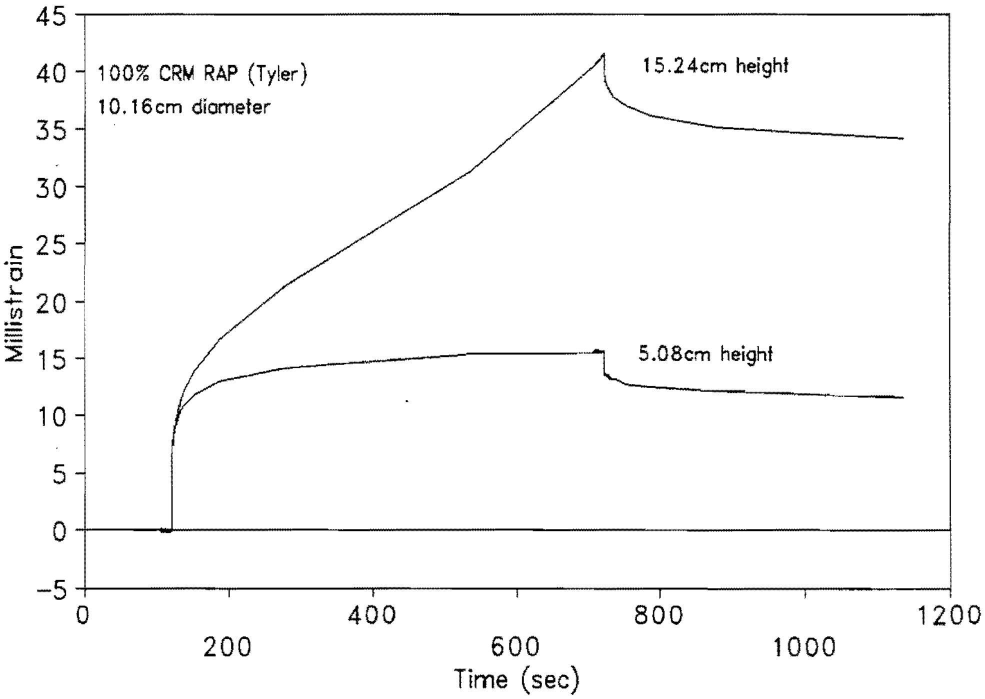

Figure B-2. Comparison of the Impact of Specimen Height on Creep Response.

Permeability

Permeability data for laboratory molded CRM asphalt specimens are shown below in Table B-14.

Table B-14. Falling Head Permeability Data for Molded CRM Asphalt Specimens.

7/9/94 5.08-cm Acrylic Tubing - Pressurized, 206.8 kPa Specimen ID: Sample 5							
Run	h1 (cm)	h2 (cm)	t (sec)	L (cm)	A (cm^2)	K (cm/s)	Average (cm/s)
1	123.9	115.6	3.90	2.228	81.07	2.516e-02	2.530e-02
2	115.6	106.8	4.58	2.228	81.07	2.446e-02	
3	106.8	98.9	4.37	2.228	81.07	2.489e-02	
4	98.9	91.0	4.54	2.228	81.07	2.595e-02	
5	91.0	85.1	3.64	2.228	81.07	2.606e-02	
7/11/94 Pressurized, 206.8 kPa Specimen ID: Sample 4							
Run	h1 (cm)	h2 (cm)	t (sec)	L (cm)	A (cm^2)	K (cm/s)	Average (cm/s)
1	163.9	155.7	6.10	2.367	81.07	1.265e-02	1.371e-02
2	155.7	149.4	4.59	2.367	81.07	1.353e-02	
3	143.9	138.5	4.10	2.367	81.07	1.402e-02	
4	138.5	133.2	4.16	2.367	81.07	1.410e-02	
5	133.2	127.9	4.28	2.367	81.07	1.426e-02	
7/12/94 Pressurized, 206.8 kPa Specimen ID: Sample 1							
Run	h1 (cm)	h2 (cm)	t (sec)	L (cm)	A (cm^2)	K (cm/s)	Average (cm/s)
1	167.9	163.0	4.76	2.259	81.07	8.928e-03	8.985e-03
2	163.0	158.0	5.00	2.259	81.07	8.940e-03	
3	158.0	154.0	3.95	2.259	81.07	9.314e-03	
4	154.0	149.6	4.60	2.259	81.07	9.041e-03	
5	149.6	144.6	5.57	2.259	81.07	8.756e-03	
6	144.6	140.5	4.62	2.259	81.07	8.933e-03	

Table B-14. (Cont.)

7/13/94
Pressurized, 206.8 kPa
Specimen ID: SML3.R1

Run	h1 (cm)	h2 (cm)	t (sec)	L (cm)	A (cm^2)	K (cm/s)	Average (cm/s)
1	156.3	155.9	83	2.377	81.07	4.661e-05	5.537e-05
2	155.8	155.4	76	2.377	81.07	5.107e-05	
3	155.3	154.8	85	2.377	81.07	5.728e-05	
4	154.7	154.2	89	2.377	81.07	5.491e-05	
5	154.0	153.4	88	2.377	81.07	6.697e-05	

7/15/94
Pressurized, 206.8 kPa
Specimen ID: SRM3.R3

Run	h1 (cm)	h2 (cm)	t (sec)	L (cm)	A (cm^2)	K (cm/s)	Average (cm/s)
1	158.8	158.5	336	2.512	81.07	8.979e-06	6.766e-06
2	158.5	158.3	302	2.512	81.07	6.670e-06	
3	158.3	158.1	416	2.512	81.07	6.062e-06	
4	158.0	157.8	304	2.512	81.07	6.648e-06	
5	157.7	157.5	370	2.512	81.07	5.472e-06	

7/16/94
Pressurized, 206.8 kPa
Specimen ID: PAVE.2

Run	h1 (cm)	h2 (cm)	t (sec)	L (cm)	A (cm^2)	K (cm/s)	Average (cm/s)
1	152.4	151.0	5.54	1.309	81.07	1.385e-03	1.403e-03
2	151.0	149.6	5.50	1.309	81.07	1.408e-03	
3	149.6	148.1	5.80	1.309	81.07	1.445e-03	
4	148.1	146.7	5.64	1.309	81.07	1.400e-03	
5	146.7	145.1	6.62	1.309	81.07	1.377e-03	

Table B-14. (Cont.)

7/19/94 Pressurized, 206.8 kPa Specimen ID: STL3							
Run	h1 (cm)	h2 (cm)	t (sec)	L (cm)	A (cm^2)	K (cm/s)	Average (cm/s)
1	165.2	164.2	80	2.495	81.07	1.203e-04	
2	164.2	163.2	90	2.495	81.07	1.076e-04	1.048e-04
3	163.2	162.4	84	2.495	81.07	9.270e-05	
4	162.4	161.6	81	2.495	81.07	9.661e-05	
5	161.6	160.7	83	2.495	81.07	1.066e-04	
8/1/94 Pressurized, 206.8 kPa Specimen ID: SHM10.R3							
Run	h1 (cm)	h2 (cm)	t (sec)	L (cm)	A (cm^2)	K (cm/s)	Average (cm/s)
1	163.1	158.6	46.9	2.217	81.07	8.400e-04	
2	158.6	157.3	15.4	2.217	81.07	7.526e-04	7.124e-04
3	157.3	156.0	15.6	2.217	81.07	7.491e-04	
4	156.0	154.7	15.8	2.217	81.07	7.458e-04	
5	152.9	152.2	15.6	2.217	81.07	4.142e-04	
6	152.2	150.9	15.8	2.217	81.07	7.645e-04	
7	150.9	149.7	15.6	2.217	81.07	7.207e-04	
8/1/94 Pressurized, 206.8 kPa Specimen ID: SLM0.R3							
Run	h1 (cm)	h2 (cm)	t (sec)	L (cm)	A (cm^2)	K (cm/s)	Average (cm/s)
1	165.0	164.0	75	2.155	81.07	1.109e-04	
2	164.0	162.9	60	2.155	81.07	1.535e-04	1.406e-04
3	162.9	161.8	60	2.155	81.07	1.546e-04	
4	161.8	160.8	60	2.155	81.07	1.414e-04	
5	160.8	159.8	60	2.155	81.07	1.423e-04	

Table B-14. (Cont.)

8/2/94 Pressurized, 206.8 kPa Specimen ID: Sample 1A							
Run	h1 (cm)	h2 (cm)	t (sec)	L (cm)	A (cm^2)	K (cm/s)	Average (cm/s)
1	173.8	169.0	7.40	2.182	81.07	5.245e-03	
2	169.0	163.6	8.20	2.182	81.07	5.488e-03	5.417e-03
3	163.6	158.6	7.94	2.182	81.07	5.418e-03	
4	158.6	153.5	8.43	2.182	81.07	5.373e-03	
5	146.3	141.4	8.49	2.182	81.07	5.561e-03	
8/2/94 Pressurized, 206.8 kPa Specimen ID: Sample 2A							
Run	h1 (cm)	h2 (cm)	t (sec)	L (cm)	A (cm^2)	K (cm/s)	Average (cm/s)
1	169.6	165.4	6.72	2.165	81.07	5.131e-03	
2	165.4	161.0	6.92	2.165	81.07	5.358e-03	5.357e-03
3	159.4	155.4	6.54	2.165	81.07	5.344e-03	
4	155.4	151.1	7.20	2.165	81.07	5.359e-03	
5	151.1	146.8	7.10	2.165	81.07	5.591e-03	

FIELD STUDIES

NDT (ARAN, FWD, Radar, Pavement Evaluation)

Table B-15 presents data from falling weight deflectometer (FWD) testing. As was expected at the time this testing was proposed, the FWD does not show any statistically significant differences between the deflections measured before and after the inlay at 55 MPa plate loading. The inlay is relatively thin and the measurements were taken within a few months of each other under similar environmental conditions, so the FWD would not be expected to show much difference in this scenario. If long term FWD monitoring had been started several years before the inlay and then continued for several years after the inlay, the FWD data might have been useful from the standpoint of providing information on the rate of change of response variables.

Mixture

Table B-16 shows the field densities obtained during construction of the overlay.

In a demonstration test, the researchers employed two relatively new techniques to study the job in San Antonio. These two tools were infrared (IR) video and ground penetrating radar (GPR). The IR video was used to study plant and laydown operations while the radar was used to study the finished in-place materials.

The CRM RAP mix design resulted in a mix that was similar in nature to a CMHB in that it appeared to be rich in asphalt and somewhat open or coarse. There was some concern that the material might be permeable to water. Since this was an inlay application with no positive drainage installation, a permeable material could result in moisture accelerated damage with eventual deteriorization of the thick binder films as well as moisture induced problems in lower layers. The material provided good surface texture for friction, but this same texture that is beneficial for friction tends to retain water and increases the time required for a given water particle to reach the pavement edge over that required to reach the edge on a pavement with the same crown but a smoother texture.

The evaluation of moisture within the pavement layers was evaluated using ground penetrating radar. Most of the radar study was performed between mile markers 551 and 553 in the westbound lane and just west of the Camp Bullis exit in the eastbound lane. At

Table B-15. Falling Weight Deflectometer Data Taken Before and After the CRM RAP Inlay.

Time[1]	Station	Load (MPa)	Df1 (mm)	Df2 (mm)	Df3 (mm)	Df4 (mm)	Df5 (mm)	Df6 (mm)	Df7 (mm)
A	0.552	55.78745	0.21825	0.08375	0.044	0.031	0.02425	0.01825	0.01375
A	1.123	57.07681	0.28275	0.14075	0.05975	0.02975	0.019	0.01275	0.00975
A	1.551	55.10484	0.2465	0.1195	0.05775	0.03425	0.02575	0.0195	0.015
A	3.55	54.28434	0.26625	0.138	0.069	0.03925	0.02725	0.02	0.01575
A	4.552	54.69114	0.22225	0.10375	0.051	0.0325	0.0265	0.019	0.01475
A	7.552	55.0221	0.21025	0.08125	0.0475	0.03375	0.027	0.0205	0.0165
B	0.552	55.87708	0.22125	0.0845	0.044	0.03125	0.02425	0.018	0.01375
B	1.072	55.73229	0.2065	0.07275	0.032	0.022	0.02	0.01475	0.01125
B	1.551	55.07037	0.249	0.12	0.0575	0.03475	0.026	0.01925	0.01475
B	3.55	54.24297	0.26975	0.13875	0.069	0.03925	0.02775	0.0205	0.01575
B	4.552	54.84283	0.22625	0.105	0.0515	0.03325	0.026	0.019	0.01475
B	7.552	54.93936	0.2135	0.08075	0.04775	0.03375	0.02675	0.0205	0.01675

[1] A=After CRM RAP inlay, B=Before inlay.

Table B-16. Field Densities Obtained During Construction of Original IH-10 Overlay.

Sample	Core #	Location (WB Lane)	Nuclear Density (kg/m^3)	Core Density (kg/m^3)	% Air Voids
15-92-537	1	471+04 2'R	2143	2098	11.4
"	2	421+84 2'R	2166	2087	11.9
"	3	369+05 2'R	2134	2038	14.0
15-92-542	4	474+64 8'R	2169	2105	10.6
"	5	454+44 8'L	2130	1988	15.5
"	6	434+59 8'L	2151	2052	12.8
15-92-574	7	286+03 2'L	2105	2185	6.8
"	8	284+60 8'L	2098	2191	6.6
"	9	278+06 4'L	2097	2105	10.3
15-92-583	10	248+42 12'L	2042	2203	6.4
"	11	238+57 4'L	2001	2145	8.9
"	12	224+06 4'L	1972	2126	9.7
15-92-584	13	165+02 8'R	2018	2055	12.4
"	14	141+62 12'R	1994	2028	13.5
"	15	97+67 12'R	1964	2111	10.0

mile marker 553 in the westbound lane, a particularly interesting study was performed. At this location, TxDOT used a water truck to flood part of the newly placed CRM RAP material. The radar was used to determine if any difference between the flooded section and the dry section could be seen. The radar was also used to check the layer thickness. Physical measurements of a core taken at this location indicated that the CRM RAP thickness (termed "layer 1" here) was approximately 71 mm and that the total asphalt layer (CRM RAP + underseal + original asphalt pavement layer, termed "layer 1 + layer 2" here) was approximately 142 mm thick. As shown in Figure B-3, the total asphalt layer thickness measurement by radar over the entire length of the section ranged from approximately 120 mm to 150 mm which correlates well with the physical measurements of the core. Figures B-4 - B-6 illustrate that the flooded section materials did allow water to penetrate all the way down to the top of the base layer. In Figure B-4, the higher dielectric beginning at approximately 60 m indicates the start of the flooded section. The dielectric remains virtually constant in the first layer (the CRM RAP layer) from the start of the wet section to the end at approximately 182 m. A similar increase in the dielectric is seen in the underlying asphalt (original IH-10 conventional asphalt material). The third layer dielectric plotted in Figure B-6 reflects the intrusion of water into the base course. This is most apparent between the 80 m and 120 m distance locations on the section.

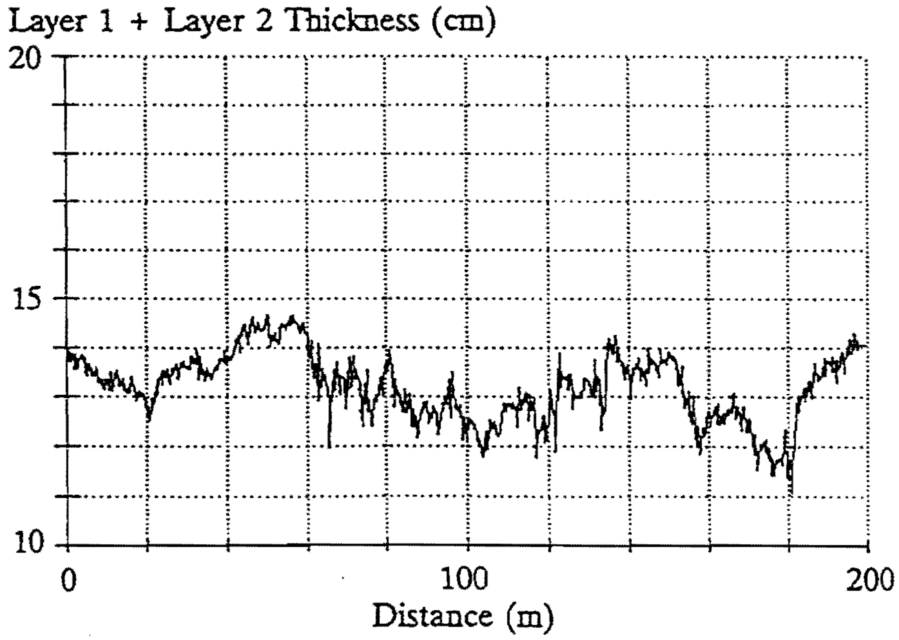

Figure B-3. Thickness of Asphalt Concrete Layer Measured by GPR.

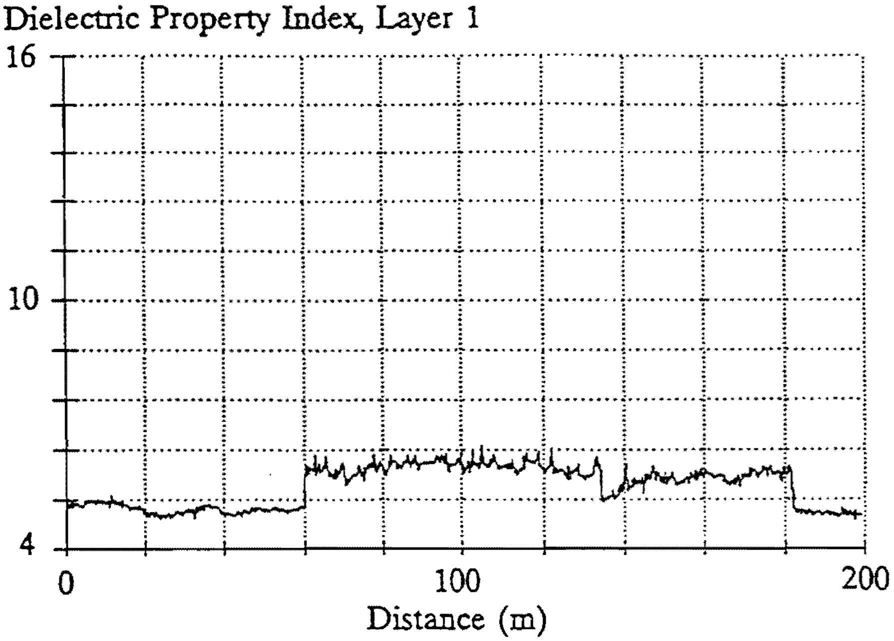

Figure B-4. Dielectric Property for Layer 1 (CRM RAP).

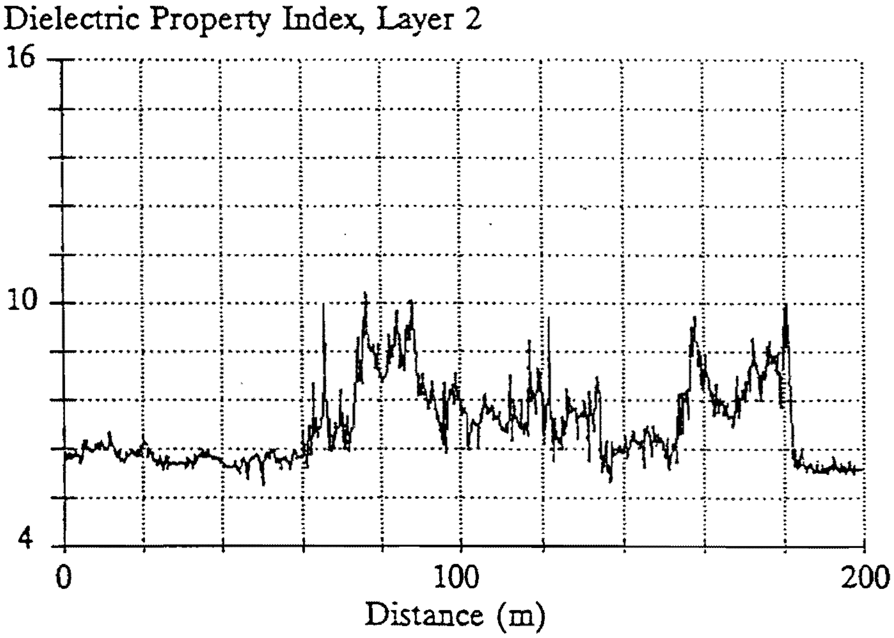

Figure B-5. Dielectric Property for Layer 2 (Original Conventional Asphalt).

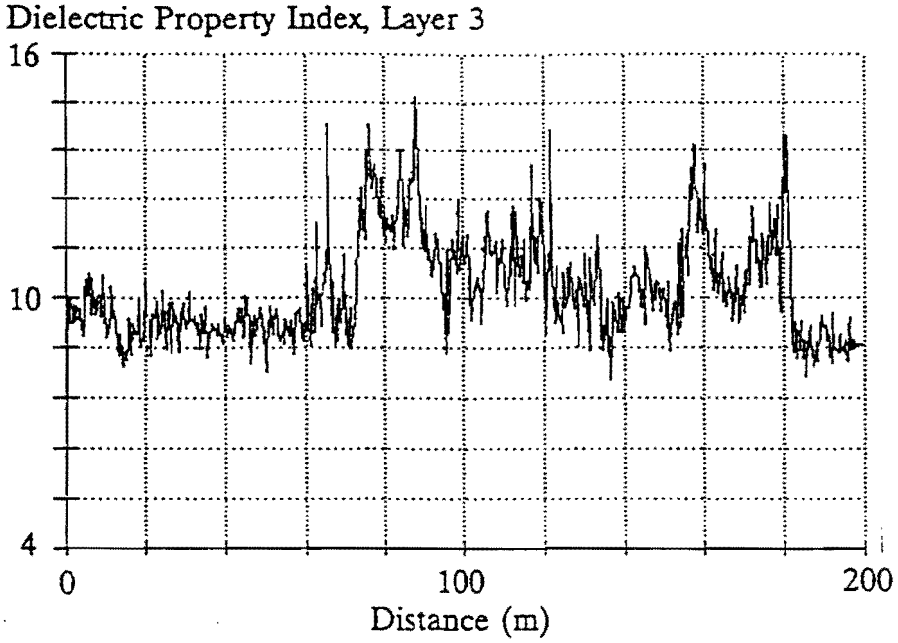

Figure B-6. Dielectric Property for Layer 3 (Base Course).

It is apparent that this permeability and air void content are not low enough to keep water from entering the pavement system. It remains to be seen how detrimental such water intrusion will be to the life of the pavement. However, it is clear that a positive drainage system would have improved its chances for a long life.

The infrared video provided interesting data on equipment and material temperature and operating characteristics. It is obvious from the videotape that the handheld IR thermometers and mercury thermometers are vastly inferior to IR video for temperature measurement of the mat. Problems with the laydown machine and the freshly laid material that were invisible to the naked eye were easily identified with IR video. Limited laboratory investigation using surface temperature measurements on CRM RAP specimens with varying percentages of RAP content had indicated that the specimens with higher rubber contents cooled faster than those with lower rubber contents (50% and 100% CRM RAP mixtures were compared). Because the haul from the plant to the construction site was so long, there was some concern that the mix would be too cool at laydown. IR video was used to track a load of material from the plant to laydown and compaction. Figures B-7 - B-9 illustrate the temperature change of this load of material. Even for this haul length, heat retention seemed to be adequate in the environmental conditions encountered during the job.

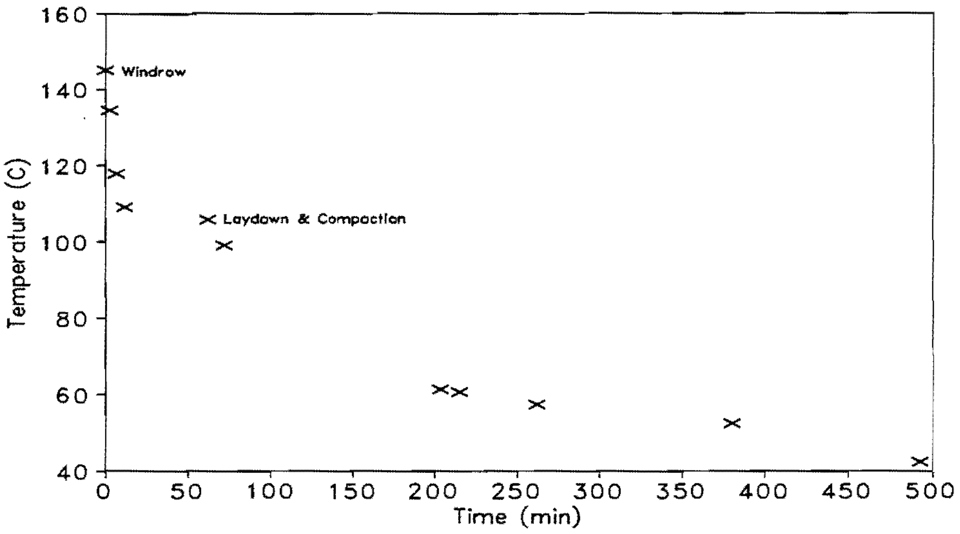

Figure B-7. IR Temperature History at Test Point 1.

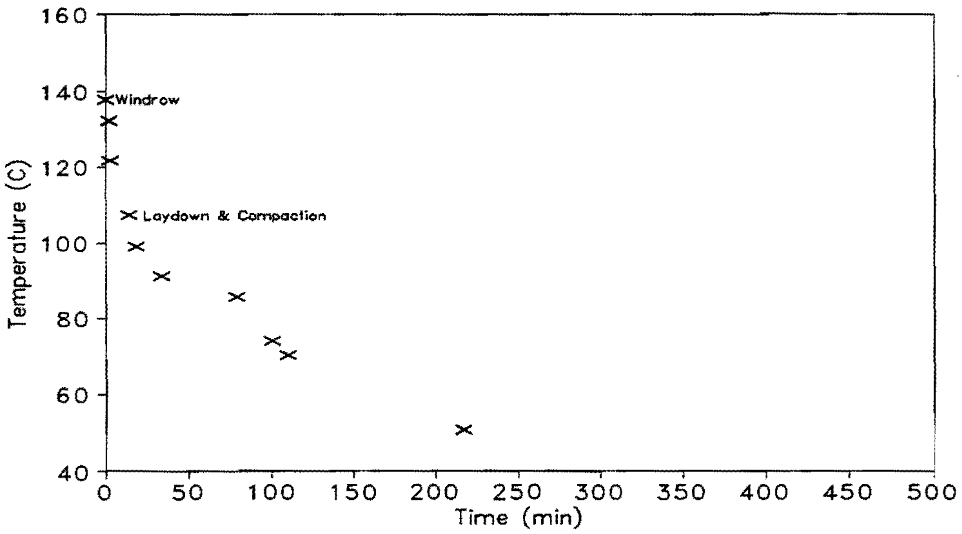

Figure B-8. IR Temperature History at Test Point 2.

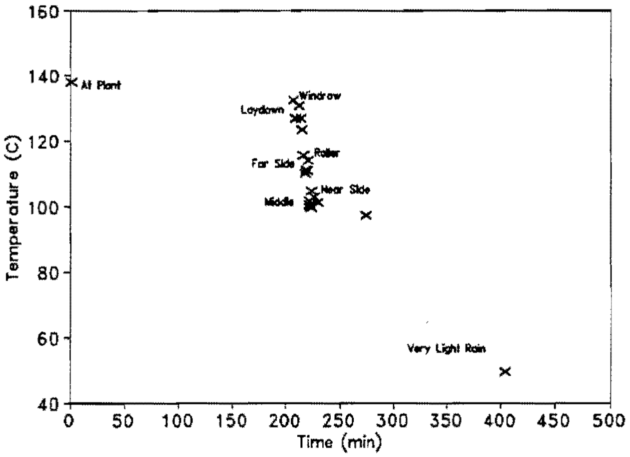

Figure B-9. IR Temperature History at Test Point 3.

Therefore, although long hauls are not recommended, they are feasible when properly undertaken.

The effect of plant variability on several different mixture parameters can be seen in Figures B-10 through B-18.

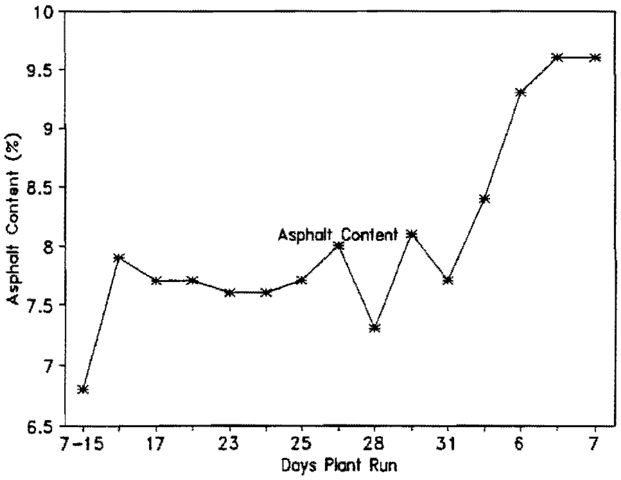

Figure B-10. Asphalt Content for Original CRM RAP Project.

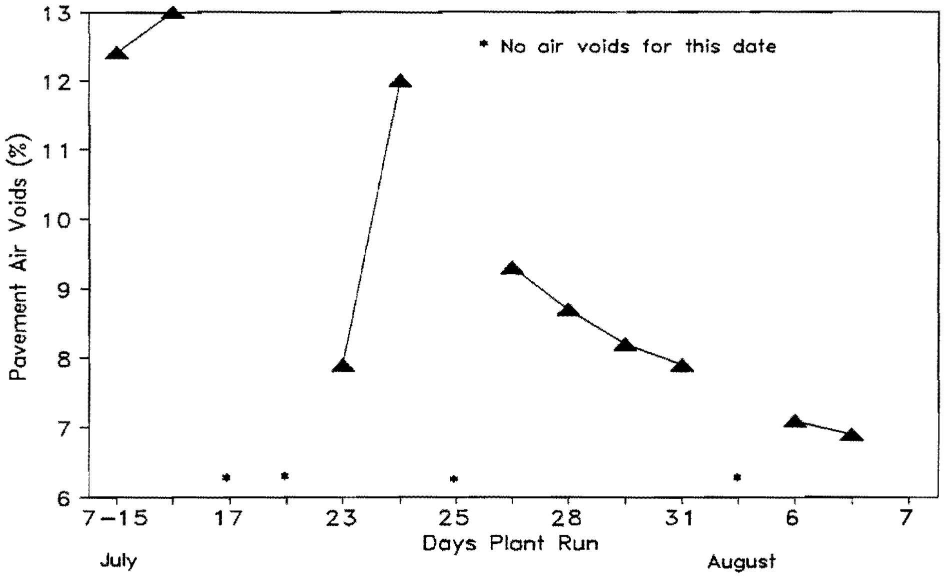

Figure B-11. Air Voids for Original CRM Project.

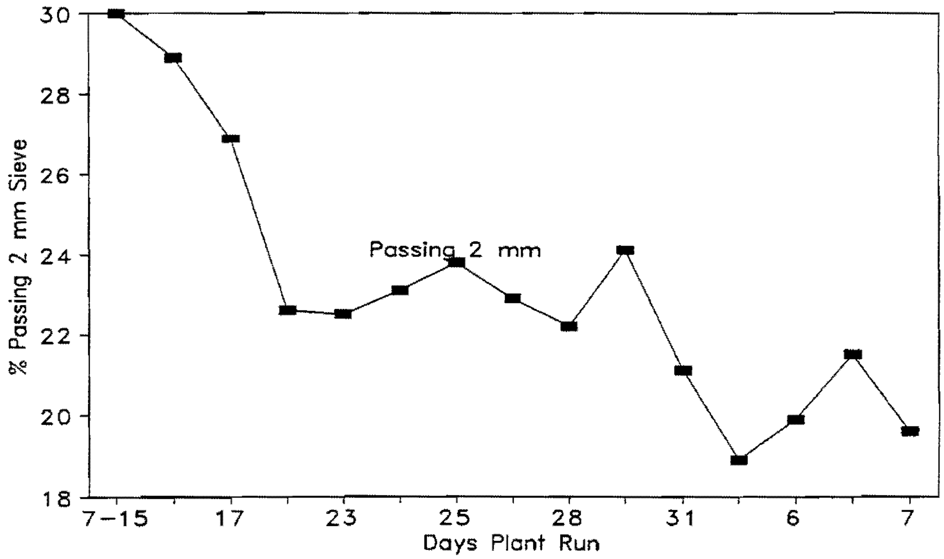

Figure B-12. Percent Passing 2 mm Sieve for Original CRM Project.

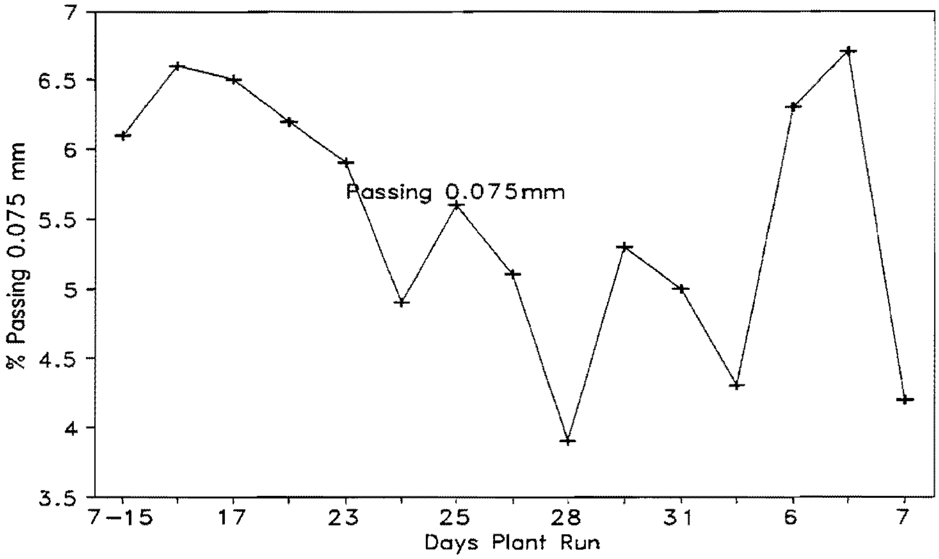

Figure B-13. Percent Passing 0.075 mm Sieve for Original CRM Project.

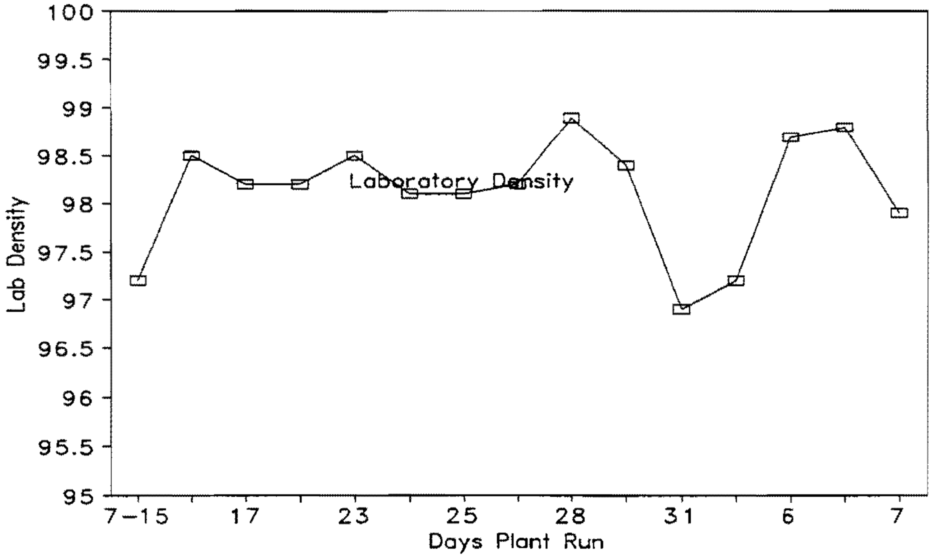

Figure B-14. Laboratory Density for Original CRM Project.

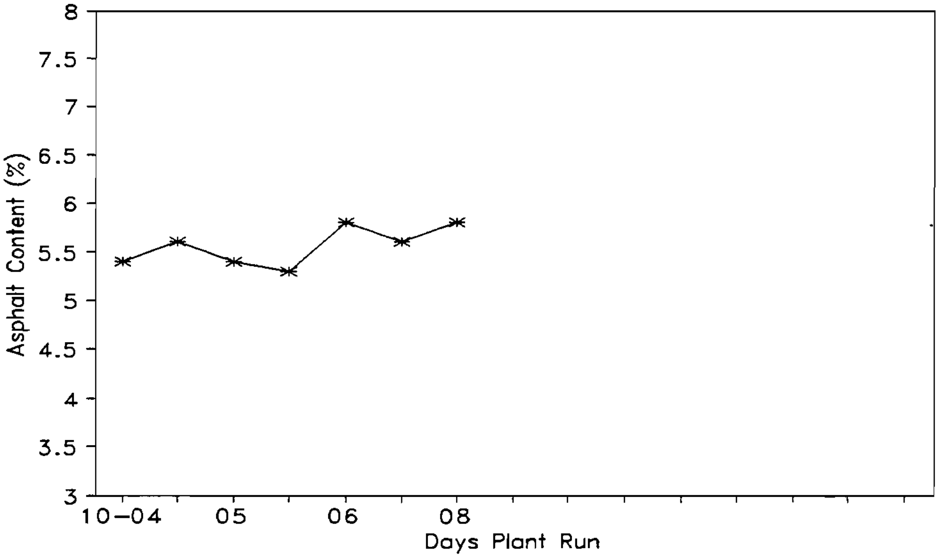

Figure B-15. Asphalt Content for CRM RAP Project (October 1993).

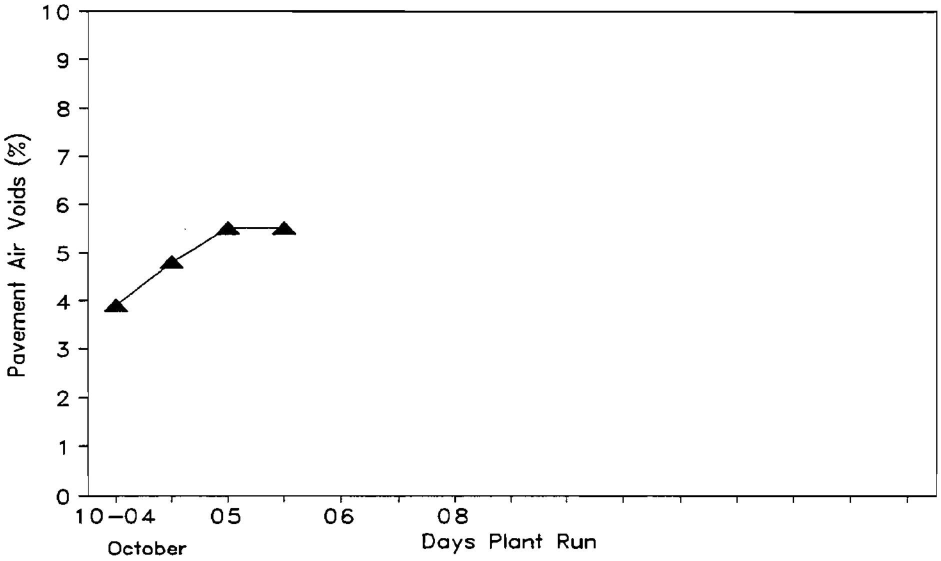

Figure B-16. Air Voids for CRM RAP Project (October 1993).

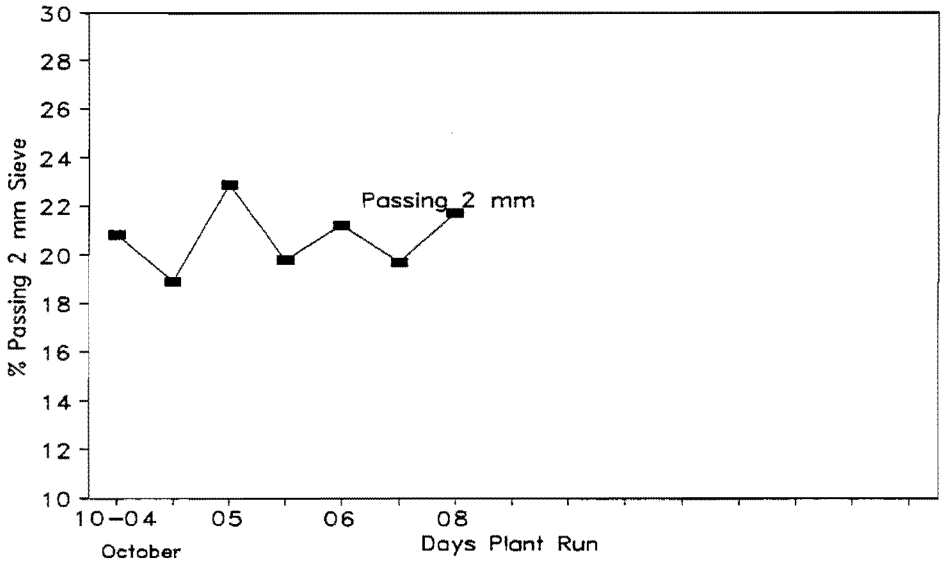

Figure B-17. Percent Passing 2 mm Sieve for CRM RAP Project (October 1993).

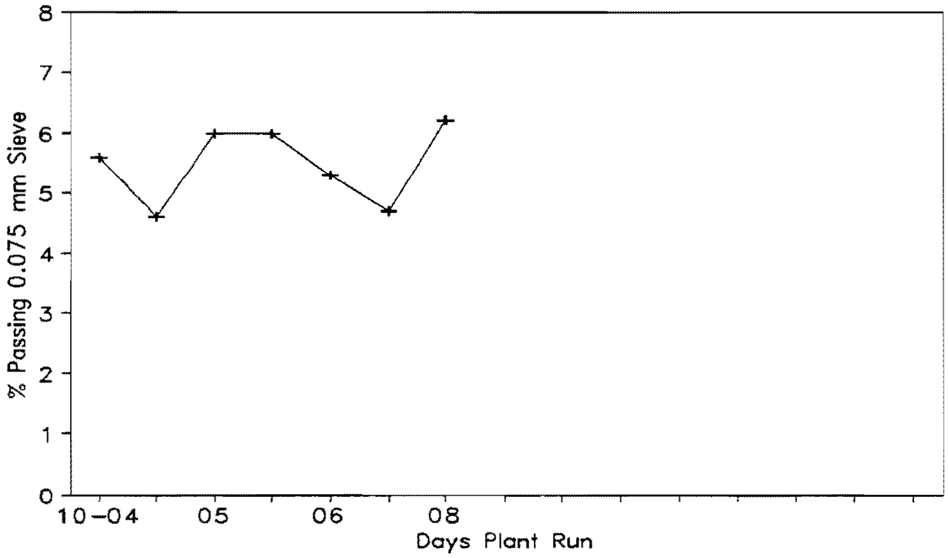

Figure B-18. Percent Passing 0.075 mm Sieve for CRM RAP Project (October 1993).

APPENDIX C - ASPHALT RUBBER RECYCLING GUIDELINES

CONSTRUCTION ALTERNATIVES

Epps et al. (1980) identify twenty-four alternatives for recycling asphalt concrete pavements. Recycling methods are generally used to improve surface distress, structural inadequacy, ride quality, and skid resistance of a pavement. The differences are primarily in the depth of the recycle, the presence or absence of heat, and the presence or absence of new binder in the recycle. These alternatives are divided into three broad categories: surface, in-place, and central plant recycling.

Surface Recycling

This technique involves reworking the surface of a pavement to a depth of less than 25 mm (1 in.). This procedure is presently the most popular form of recycling to address a wide variety of pavement distress, including rutting, raveling, flushing, and corrugation. However, it has only a very limited effectiveness in repairing severely rutted roads or in significantly increasing the load-carrying capacity of the roadway.

The equipment used in recycling techniques requiring heat are heater planers, heater scarifiers, and hot millers. Cold recycling uses cold planers and cold millers. A guide to equipment alternatives is provided below.

Heater Planer

The heater planer maintains longitudinal grade and transverse cross slope, and is ideal for heating and planing a pavement prior to overlay. Also, to correct poor skid resistance in flushing or bleeding asphalt rubber mixes, the heater is used to heat the pavement while a seal-coat aggregate is spread and imbedded into the distressed pavement with a steel-wheel roller.

Heater Scarifiers

Heater scarifiers remove pavement surface regularities. When a new wearing course is desired after scarification, the heater scarifier may improve the bond between old pavement and new asphalt rubber overlay.

Hot and Cold Millers

Milling operations improve the surface texture of the roadway. This improved surface texture increases skid resistance as well as the shear strength between the old surface and a new overlay. The millings can be treated either in-place or at a central plant, and can be used for unstabilized base courses or stabilized base and surface courses. With asphalt rubber mixes, the use of heat often results in milled rubber particles sticking to equipment, so cold milling provides a less problematic alternative.

Cold Planers

Cold planing is commonly used to remove corrugations and improperly constructed chip seals. While it does not provide a surface appearance as smooth as heater planing, heat-free planing does not result in high emission levels of air pollutants, making it the most environmentally safe alternative.

In-Place Surface Recycling

In-place surface recycling has resulted from the development of pulverizing equipment and processing techniques, and has a major advantage of improving the load-carrying capability of the pavement without changes in the geometry of the roadway. Two basic approaches can be used, depending on the thickness of the asphalt concrete surface. If the surface is about 125 mm (5 in.) thick or less, pulverization equipment can be used without preliminary ripping and breaking. For a thicker surface, scarifiers, dozers, or compactors may be used for the initial breakup, followed by a pulverization process.

The disadvantages of in-place surface recycling are that quality control is not as good as with central plant operations, and traffic disruption may be relatively high.

Central-Plant Recycling

The pavement is ripped and broken to a size to be received by the primary crusher prior to loading onto the haul units. The pavement can be reduced in size in-place and then hauled to the central plant, or can be removed from the site and then sized using other equipment. When reused at the plant, the recycled hot mix process involves the use of additional heat and recycling agents.

Central-Plant sizing can be performed with conventional, fixed, and portable crushing and screening equipment. It maintains good quality control, although at a high cost. Selection of central plant recycling alternatives depends on the availability of plant equipment, the need for structural improvement, and the distance of haul to new aggregate and existing plants. With CRM asphalt mixes, the plant should have the necessary modifications to allow for sticky rubber at high mixing temperatures. The following guidelines are owed in large part to recommendations taken from the FHWA Crumb Rubber Modifier workshop of 1993.

STORAGE AND HANDLING OF CRM

Crumb rubber is commonly packaged in polyethylene bags of approximately 27 kg (60 lbs), or larger bulk of approximately one ton in capacity. Both forms of rubber packaging are pelletized and require tied-down plastic sheeting for additional moisture protection during storage. Crumb rubber pellets are handled with forklifts and standard conveyor belts are used for polyethylene bags. The rubber is generally fed into the weigh hopper or pug mill of a batch plant or the RAP feed system on a drum plant. Where graded CRM is used, dispersion of the rubber throughout the hot mix must be ensured.

HOT MIX EQUIPMENT AND PRODUCTION

A conventional hot mix asphalt mixing facility can be modified with automatic controls that coordinate proportioning, timing, and discharge of the mixture. The facility shall be capable of uniformly feeding and measuring the amount of crumb rubber placed into the mixing chamber, with the capability to heat binder supply lines.

When a drum plant is used, the metering equipment is hooked up by installing a two- or three-way valve in the feed line on the output side of the asphalt pump. The metering equipment is then plumbed to the valve to feed the asphalt rubber binder accurately to the hot mix plant. Special pumps are used to prevent damage to conventional pumps.

When a batch plant is used, the valve is installed directly onto the supply line leading to the weigh bucket and the metering equipment is plumbed as described above.

Asphalt rubber shall not be transported on rubber belts. Cold CRM RAP may be transported on rubber belts.

The crumb rubber shall not be added to the aggregate cold feed system, but will be added beyond the drying and heating section of the mixing chamber. CRM RAP must be crushed to a size that can be adequately heated in the drum. This is usually done at the beginning of the feed system to the RAP collar.

PAVING EQUIPMENT

Distributor

The distributor shall be capable of uniformly applying the asphalt rubber binder at the specified temperature and application rate, while providing continuous circulation of the binder in the tank for homogeneity until it is metered into the hot mix facility mixing chamber.

Aggregate Spreader

The aggregate spreader shall be self-propelled and of sufficient capacity to apply the aggregate within the specified time.

Hauling Equipment and Rollers

The hauling equipment and compaction rollers may be thinly coated with a light application of non-petroleum based wetting agent such as soapy water or silicone emulsion to reduce sticking of the mixture to the equipment. Oiling the surfaces with kerosene or diesel fuel will not be permitted. The rollers shall be steel-wheeled and capable of reversing without backlash. Each tire shall be inflated to a minimum of 700 kPa (100 psi) and carry a minimum of 1,360 kg (3,000 lb). Pneumatic-tired rollers may only be used with surface treatments containing asphalt rubber binder, and with mixtures having up to 30% CRM RAP content. Beyond 30% CRM RAP, the use of rubber-tired rollers should be considered experimental until sufficient data indicates otherwise.

OTHER CONSTRUCTION GUIDELINES

Weather

For a compacted thickness less than 38.1 mm (1.5 in.), the application of an asphalt rubber surface treatment shall only be permitted at a minimum surface and ambient air temperature of 15°C (60°F). For a compacted thickness 3.81 cm (1.5 in.) and greater, the minimum temperature requirements shall be 10°C (50°F).

Delays

When a delay in surface treatment application occurs, the asphalt rubber binder shall be allowed to cool. Just prior to use, the asphalt rubber shall be reheated to the specified mixing temperature, thoroughly mixed, and the viscosity checked. The asphalt rubber binder shall be rejected if the viscosity fails specifications.

Application of the Binder

The binder viscosity may be adjusted to improve spray application by adding a kerosene diluent. This should occur in the distributor immediately prior to the spray application. When a diluent is used, the binder temperature for spraying shall not exceed 150°C (300°F).

LAYDOWN AND COMPACTION

Temperature

The following temperature ranges are common for asphalt rubber hot-mix applications:

(a)	Hot plant mixing temperature	138 to 154°C (280 to 310°F)
(b)	Laydown temperature	132 to 149°C (270 to 300°F)
(c)	Compaction temperature	above 115°C (240°F)

At lower CRM RAP contents (e.g., 30%), temperatures are similar to conventional mixtures without rubber.

Breakdown Rolling

Use two to four passes in the vibratory mode (full width of mat) with a double drum steel wheel roller, high frequency , low amplitude. Pneumatic rollers should NOT be used on new (100%) CRM pavements. Steel drums should be equipped with pads and a watering system.

RELATED PUBLICATIONS

ASTM C29, Test Method for Unit Weight and Voids in Aggregate.

ASTM C127, Test Method for Specific Gravity and Absorption of Coarse Aggregate.

ASTM C128, Test Method for Specific Gravity and Absorption of Fine Aggregate.

ASTM D5, Penetration of Bituminous Materials.

ASTM D1856, Recovery of Asphalt from Solution by Abson Method.

ASTM D4887, Practice for Preparation of Viscosity Blends for Hot Recycled Bituminous Materials.

ASTM D4791, Test Method for Flat or Elongated Particles in Coarse Aggregate.

Chehovits, J.G., "Binder Design Procedures," Crumb Rubber Modifier Workshop Notes, Design Procedures and Construction Practices, FHWA, Session 9.

Cooper, K.E., S.F. Brown, and G.R. Pooley, "The Design of Aggregate Gradings for Asphalt Basecourses," Asphalt Paving Technology, AAPT, Vol. 54, pp. 324-345.

Epps, J.A., "Uses of Recycled Rubber Tires in Highways," NCHRP Synthesis of Highway Practice 198, 1994.

Lovering, W. R. and J. Matthews, "Design and Control of Asphalt Mixes," Institute of Transportation Studies, Course Notes, University of California, Berkeley, May 1978.

Roberts, F.L., P.S. Kandhal, E.R. Brown, D-Y Lee, and T.W. Kennedy, *Hot Mix Asphalt Materials, Mixture Design, and Construction,* NAPA, 1991.

AASHTO TP7, Standard Method for Determining the Permanent Deformation and Fatigue Cracking Characteristics of Hot Mix Asphalt (HMA) Using the Simple Shear Test (SST) Device.

Tex-200-F, Sieve Analysis of Fine and Coarse Aggregate.

Tex-201-F, Bulk Specific Gravity and Water Absorption of Aggregate.

Tex-206-F, Method of Compacting Test Specimens of Bituminous Mixtures.

Tex-210-F, Determination of Asphalt Content of Bituminous Mixtures by Extraction.

Tex-211-F, Recovery of Asphalt from Bituminous Mixtures by the Abson Process.

Tex-224-F, Determination of Flakiness Index.

Tex-231-F, Static Creep Test.

Tex-232-F, Mixture Design Procedure for Crumb Rubber Modified Asphaltic Concrete.

Tex-234-F, Mixture Design Procedure for Coarse Matrix High Binder Asphaltic Concrete.

Tex-404-A, Determination for Unit Weight of Aggregate.

Tex-405-A, Determination for Percent Solids and Voids in Aggregate for Concrete.

Tex-410-A, Abrasion of Coarse Aggregate by Use of the Los Angeles Machine.

Tex-502-C, Test for Penetration of Bituminous Materials.

Texas Construction Specification Item 300, Asphalts, Oils and Emulsions.

Texas Construction Specification Item 340, Hot Mix Asphaltic Concrete Pavement.

Tunnicliff, D.G., "A Review of Mineral Filler," Proceedings of The Association of Asphalt Paving Technologists, Volume 31, 1962, pp. 118-150.

SCOPE

The goal of this procedure is to provide guidance for the design of bituminous mixtures containing crumb rubber reclaimed asphalt pavement (RAP). It should be used in conjunction with applicable Texas Construction Specifications such as Items 300 and 340. In most cases, the RAP is available because the pavement is performing inadequately. This inadequate performance is usually caused either by improper mixture design or improper construction techniques. Both sources of difficulty are addressed in the procedure.

The first objective is to design a mixture which will ensure that the load is carried by the stone skeleton. For aggregate blending purposes, meeting this objective implies that any rubber particle greater in size than the asphalt film thickness should be considered as part of the aggregate. Note that the previous statement does not necessarily mean that the rubber influences the behavior of the mix in the same way as the aggregate; rather the intent is to provide sufficient room within the aggregate skeleton for the rubber to act primarily as a binder modifier. In order to facilitate portability of the procedure between the laboratory and the plant, the procedure emphasizes RAP and virgin material stockpile characteristics at the plant. Therefore, the second objective is to ensure that the aggregate in the largest stockpile to be used in the mixture actually participates in carrying the load. This objective can be generalized to all stockpiles to be used in the mix if more than one stockpile is used. The third objective is to provide for adequate binder film thickness for the intended use of the material without allowing draindown. The design procedure is intended for use with standard hot mix applications as given in the example, but can be easily modified for use in special applications such as open-graded drainage or friction courses by choosing the correct stockpile characteristics and employing crumb rubber to reduce draindown and possibly oxidation problems. Therefore, in standard mix applications using this procedure, it is expected that no rubber will be added with the new material; the only rubber in the mix will be that already present in the RAP. For open-graded designs, however, additional crumb rubber may be necessary. In the latter case, it is anticipated that the wet process will be used to incorporate the rubber.

The final objective is to provide a binder that has the desired performance characteristics. This can be done through asphalt binder blending tests. Successfully reaching this objective should not affect the aggregate blending calculations. However, there is one case in which it will affect the aggregate blending calculations. That case is the one in which the addition of virgin binder and/or rejuvenator required to reach the desired asphalt binder performance characteristics overfills the available voids. In the event this problem arises, the recommended actions are, in order of preference, (1) decrease the RAP content, (2) decrease the viscosity of the virgin binder/rejuvenator, and/or (3) use a virgin stockpile that has a gradation favoring an increased asphalt content but still meets the stone-on-stone contact requirements.

The philosophy adopted in this procedure implies that there is no specified minimum or maximum RAP content in the final mix. This means that, for example, an 80% RAP content may result from the design. In practical terms, such a high RAP content requires that conventional plant production rates must be reduced if the RAP is either wet or not sufficiently crushed. Relatively new recycling equipment[1] has been or is being developed to address the challenges presented by higher RAP content mixtures.

EVALUATE RAP

1. Prepare specimens of the RAP (100% RAP, no modification to the reclaimed material). Evaluate these specimens with the specifications and design criteria of Tex-231-F and Tex-232-F. If stripping is suspected, evaluate the RAP using Tex-530-C and Tex-531-C as well.[2] If the RAP passes these criteria, it is assumed that the material is acceptable to TxDOT and that the reason the material failed was because of improper construction techniques. In this case, it is unnecessary to modify the RAP prior to reuse; a 100% RAP recycle is feasible (assuming any moisture problems can be overcome in the plant, and assuming that any underlying pavement structural problems have been corrected), and no further testing is required under this procedure. However, if the RAP does not meet current TxDOT criteria and specifications, the remainder of this procedure must be accomplished.

2. Obtain an approximate asphalt content (Tex-210-F), crumb rubber size, and sieve analysis of the RAP (Tex-200-F).[3] When performing the extraction of Tex-210-F, pay particular attention to the notes on crumb rubber. It may be necessary to add steps to recover the rubber as well. This is usually done by floating the rubber using a sodium

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[1] *e.g.* RAPMaster, Pyrotech, Cyclean.

[2] Recent advances in more fundamental tests for bond characteristics in the presence of fluids such as water will probably result in an improved method that will replace these test methods in the near future.

[3] It has been found (*e.g.*, *Epps 1994* p. 30, as well as in study 1333) that an accurate measure of asphalt and rubber content and properties is not possible with most current solvent extraction tests. This is thought to be a result of the interaction of the solvent with the rubber and asphalt and is related to the swelling of the rubber as well as small quantities of rubber going into solution in the asphalt over time. Therefore, asphalt and rubber contents and properties obtained through solvent extraction methods should be considered to be approximate. Performance based tests and specifications refine this approximation. It has also been found in California that nuclear gauges should not be used for determination of total binder content (*Epps 1994*).

bromide solution. However, citrus terpene has been used for floating with some success. After extraction, the asphalt should be recovered (Tex-211-F). Reblend the recovered asphalt and floated rubber particles. At this point, the normal procedure would be to measure the viscosity of the recovered asphalt rubber blend (Tex-528-C) for use in viscosity blending (ASTM D4887).[4] However, this viscosity measurement may not be possible for some blends, and it has already been shown that current extraction procedures alter the characteristics of the asphalt and rubber. Therefore, simply save the recovered and reblended CRM asphalt binder for later use in blending tests. Enough binder should be extracted, recovered, and reblended with the recovered rubber to perform 4 penetrations at 25°C (110°F), and 1 additional penetration at 35°C (160°F). Finally, one additional extraction should be performed to obtain a sample of aggregate and rubber. Recovery of this asphalt is not necessary, but the rubber must be recovered as part of the aggregate for use in the vibration tests later in the procedure.

3. Perform a sieve analysis on the aggregate from the extraction. Using the clean aggregate (no rubber) from the extraction, determine the RAP size at 50% passing (*i.e.*, D_{50} size) by linear interpolation between the two sieve sizes on either side of the 50% passing mark.

4. Measure the dry bulk specific gravity and absorption (Tex-201-F, ASTM C127, C128) of the extracted aggregate-rubber blend.

5. For purposes of this procedure, the crumb rubber is assumed to have a specific gravity of 1.15, and a surface area of 150 cm^2/g (60 in^2/g) (see gas absorption results given by *Chehovits 1993*). However, if 100% of the crumb rubber will not pass through a 0.635 cm (.25 in) sieve, a sieve analysis should be conducted on the rubber and the surface area should be either calculated using the surface area factor methodology or measured.

6. Measure the penetration of the recovered asphalt rubber blend at 25°C (110°F).

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[4] It is expected that in the near future, performance grading type analyses using rheometers will become preferred methods of determining blending needs. However, some rheometers do not currently have the capacity to test crumb rubber modified asphalt binders, and procedures using rheometers that do have that capacity at present are still in the development stage (*e.g.*, plate spacings on torsional shear rheometers is still somewhat subjective).

EVALUATE VIRGIN MATERIAL

1. Using the sieve analyses from the plant stockpiles that will be used on the job, select the stockpile(s) of virgin material having a D_{50} size that is greater than the D_{50} size of the RAP. For open graded applications, it may be necessary to select a stockpile with even larger materials instead.

2. Measure the dry bulk specific gravities and absorption (Tex-201-F, ASTM C127, C128) of each stockpile of material that may be a candidate for use in the final bituminous mixture.

3. Measure the penetration of the available potential rejuvenating agents and/or virgin asphalts at 25°C (110°F).[5] Of the available potential rejuvenating agents and virgin asphalts, it is thought that those having the higher aromatic-to-saturate ratios and lower asphaltene contents from a Corbett analysis of the asphalt fractions will provide lower hardening susceptibility.

DESIGN NEW BLEND

1. Perform the following modified tests using the LA Abrasion (or ball mill) equipment if available.

(a) Wash the largest stockpile material on a 9.5 mm (3/8 in) sieve and dry the +9.5mm (3/8 in) material. Run the LA abrasion using the steel balls. Wash the abraded material on the 9.5 mm (3/8 in) sieve. Compute the modified LA abrasion. This portion of the procedure is a surrogate test for a fracture mechanics type test on the large aggregate. It is intended to quantify the friability of the material and give some qualitative indication

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[5] Potential rejuvenators include a wide range of materials such as low viscosity asphalts and asphalts which have viscosities that have been reduced by the addition of proprietary solutions. In this procedure, the term *rejuvenating agents* refers to additives such as these proprietary solutions or even solutions containing crumb rubber. Asphalts, regardless of grade, are considered to be *virgin asphalt cements* only if they contain no additives or modifiers.

of any tendency toward breakdown of the larger particles under compaction equipment or traffic.

(b) Combine the stockpiles so as to give a gradation curve that falls between the 0.5 and 0.7 Fuller power curve gradation (the power curve formula is given in Equation C-3 later in the procedure). Split the sample in half and perform a sieve analysis on one of these two samples. Take the other half and run 5 minutes in the LA machine *without* the steel balls (the wet ball mill Tex-116-E *without* the water and *without* the balls is an acceptable substitute for the LA machine). Sieve the abraded material.[6] This portion of the procedure is intended to simulate the action of the hot mix plant. The change in gradation from the original to the abraded material will affect asphalt content.

2. For each stockpile of material and for the aggregate-rubber blend extracted from the RAP, perform the following measurements on dry aggregate samples.

(a) Vibrate the dry aggregate in a suitably sized container (*e.g.*, a unit weight bucket) that has a weighted lid which will follow the material as it rearranges its particle orientation to a more dense state. If a vibratory table is not available, use a tamping or jigging procedure as in ASTM C29 (Tex-404-A[7]), or a high capacity sieve shaker may suffice. At the end of the densification process, measure and record the height of the lid at three points 120° apart. Remove the lid and loosen the aggregate. Repeat the vibration and measurements two more times. After the third vibration test, do not loosen the material, simply remove the lid and, if a water measurement is desired (optional for this procedure), fill the container with water up to the average level of the irregular aggregate surface. Weigh the container, aggregate, and water. Compute the weight of the water by subtraction using the data from step (a) above. Compute the volume of this water. Compute the volume taken up by the aggregate plus air and the volume of the air in the densified state using the previously measured dry bulk specific gravity of the aggregate and the average of

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[6] Recovering the material from the LA machine implies that the door on the drum is essentially air tight so that fines are not lost during the test. This often requires modification to the door (*e.g.*, weatherstrip application).

[7] Note that this procedure differs from Tex-404-A in that a full measure is not required because measurements are taken of the compacted height of the specimen.

the 9 height measurements. Record both the volume of water (if measured) and the volume of air (from height measurements).

The intent is to fill as much of the available volume as possible with the next smaller aggregate size stockpile and continue that process down through the stockpiles. However, before this next smaller stockpile can be used, it is necessary to determine if the largest particle size in this stockpile will separate the particles of the large stockpile and tend to decrease the effectiveness of this stockpile in the stone skeleton. Based on computations of available void size using several geometric configurations (*e.g.,* spherical particles in a hexagonal close packed configuration, elongated particles in a triangular configuration), the following guidelines for acceptability of the aggregate in the next smaller size stockpile, D_{MAXi}, were formulated. For uncrushed, rounded gravel type aggregates, the size at 50% passing (*i.e.,* D_{50} size) of the second stockpile must be equal to, or smaller than, 0.15 times the D_{50} size of the largest size in the first (larger) stockpile. For cubical (ratio of all lengths are 1:1) aggregate with at least one crushed face, the D_{50} size of the second stockpile must be equal to, or smaller than, 0.29 times the D_{50} size in the first (larger) stockpile. For elongated particles (ASTM D4791 uses 2:1 as the length to width ratio), the D_{50} size of the second stockpile must be equal to, or smaller than, 0.40 times the D_{50} size in the first (larger) stockpile. For this test procedure, it is adequate to determine the prevailing shape in the largest stockpile subjectively, and to use the factor corresponding to that shape for calculations involving all other stockpiles (in the future, a relationship between this factor and fractal dimensions of the aggregate from digitized video imaging should be developed). The 0.40 factor is used in the example. Note that D_{50} and D_{MAXi} are rounded to two decimal places in cm units before the decision to accept or reject the stockpile is made.

Each stockpile that meets the size restrictions is then used in the proportioning process. The proportioning can be done on a volumetric or on a weight basis. Each size fraction is adjusted for differences between the laboratory and the plant during the final blend gradation computations. In the research program, this adjustment was made by computing the ratio of the percent passing a particular size *after* LA abrasion to that passing *prior* to LA abrasion. If measurements are not available to compute this ratio, the following equation may be used to approximate the ratio. Note that this equation was developed for a limestone and bank run sand material and may not be representative for materials that do not have comparable LA abrasion values (on the order of 21% was measured for the limestone). The equation is valid for $S \leq 5.08$ cm (use a ratio of 1.0 for larger particles unless measurements indicate a larger value):

$$R_{lp} = 1.242\ (5.08 - S_{max} + S)^{-0.133} \tag{C-1}$$

where S is the square sieve opening size in cm, S_{max} is the size of the largest sieve that has less than 100% passing (all sieves having 100% passing are arbitrarily assigned a value of $R_{lp}=1.0$), and R_{lp} is the lab-to-plant adjustment ratio.

3. After the acceptable stockpile(s) have been used to fill the open volume, the remaining free volume is that available for binder assuming nonabsorptive aggregate and zero air voids. The initial choice of binder content is based on surface area factors using Equation C-2. Later in the design process, these surface area factors are applied to the percent retained as modified by the LA Abrasion tests (ratio of percent passing each size after abrasion to percent passing before abrasion) to obtain the surface area of the size fraction under study. Surface area factors have been computed on the basis of a cube with the same dimensions as the square sieve opening, a sphere having a diameter equal to the opening size, and a prolate spheroid (football shape) with the long axis along (but shorter than) the diagonal measurement for the square sieve opening size. The surface area factor formula for the cube and sphere is the same and can be obtained by the very simple formula

$$SAF = \frac{6}{S\gamma} \tag{C-2}$$

where SAF is the surface area factor in cm^2/g, and γ is the unit weight of the aggregate in g/cm^3. A tabular summary of the surface area factors and lab to plant adjustment ratios based on the equations presented above is given in Table C-1.

The asphalt content is determined by specifying a desired film thickness. The desired film thickness is assumed to be a function of the gradation (and rubber content). Therefore, one must compute the exponent of the gradation resulting from the stockpile blending

Table C-1. Tabular Summary of Equations C-1 and C-2 (γ=2.65 g/cm^3, S_{max}=0.9525 cm).

Sieve Size, S	SAF	R_{lp}
3.81cm (1.5in)	0.59	1.00
3.175cm (1.25in)	0.71	1.00
2.54cm (1.0in)	0.89	1.00
2.2225cm (0.875in)	1.02	1.00
1.5875cm (0.625in)	1.43	1.00
1.27cm (0.5in)	1.78	1.00
0.9525cm (0.375in)	2.38	1.00
0.635cm (0.25in)	3.57	1.01
0.475cm (#4)	4.77	1.01
0.20cm (#10)	11.32	1.02
0.0425cm (#40)	53.27	1.03
0.018cm (#80)	125.79	1.03
0.0075cm (#200)	301.89	1.03

procedure, and then interpolate using 7 microns for 0.45 power and 30 microns for 0.9 power as initial guidelines. The equation used for determining the exponent from regression analysis may be expressed as follows

$$P = 100 \left(\frac{d}{D}\right)^n \tag{C-3}$$

where P is the percent passing a sieve with opening size d, and D is the maximum aggregate size for the entire blend. A final addition to the computed asphalt content is made by correcting for water absorption observed during aggregate specific gravity measurements (alternatively, this correction can be made using data obtained during the CKE test, if performed). After computing the asphalt content, additional specimens should be compacted at the computed asphalt content ±0.8%. The asphalt contents used in this procedure are percents by weight of the aggregate. Note that the computed asphalt content probably will result in more or less asphalt volume than the remaining free volume calculated earlier when the aggregate stockpile blending operation is complete. This is expected because of the approximate nature of the computations (*e.g.*, no integration is performed for individual stockpile gradation and shape effects; air void content is not part of the design computation at this time, and the selection of film thickness is somewhat subjective). If the computed asphalt content minus 0.8% results in an asphalt volume that is greater than the available free volume, it is suggested that a fourth asphalt content be included in the set of compacted specimens. The fourth asphalt quantity would be (a) that which fills the available remaining free volume while allowing for the desired air voids, or (b) the computed asphalt content *minus* 1.6%, whichever is greater. It is suggested that agencies retain data on available free volume, asphalt volume (both corrected and uncorrected for absorption), and air voids of the compacted mix for use in developing a technique to reduce the number of compacted mixtures in the mix design process if air void content is specified.

4. The following data and computations are provided as an example of the process. Table C-2 summarizes the basic data on the aggregates and the measurements from vibration tests with these aggregates. The volume calculation given in this table is computed after vibrating the dry material in a suitable container. The interior dimensions of the container are used in conjunction with the measured thickness of a surcharge plate (resulting in approximately 1-2 kPa pressure simply to ensure that the vibration causes compaction and the weight does not allow decompaction) to compute the volume of the material below the surcharge after vibration (i.e., the average of three distance measurements from the top of the container to the top of the surcharge plus the surcharge thickness, subtracted from the total depth of the container when empty is the height of the aggregate plus air. Multiply the height of the aggregate plus air by the internal cross sectional area of the container to obtain the volume measurement). The weight of water listed in the table is for an alternative method of measuring the void volume by filling to the average level of the aggregate and computing the volume corresponding to the weight of water required to fill to this level. The water method should not be used unless a correction is made for absorption in the computations. The gradations of the individual components of the mix are given in Table C-3.

Table C-4 presents the results of the fundamental calculations required to define the stockpile blending proportions. First, the D_{50} size is computed for each stockpile by linearly

Table C-2. Example of Aggregate Data.

Stockpile	Bulk Specific Gravity[1]	% Absorption[1]	Volume[2] (cm^3)	Weights (g)
1	2.630	1.91	Total: 3,027.65 Aggregate: 1,689.35 Air: 1,338.30	Aggregate: 4,443 Water: 1,395
RAP	2.579	1.11	Total: 2,028.16 Aggregate: 1,401.32 Air: 626.84	Aggregate: 3,614 Water: 601

[1] Average of 3 measurements.
[2] Average of 9 height measurements used in total volume computation (3 measurements after each of 3 vibration tests)

Table C-3. Component Material Gradations.

Sieve Size (cm)	Percent Passing		
	Stockpile 1	RAP	Rubber
3.81	100.00	100.00	100.00
3.175	100.00	100.00	100.00
2.54	100.00	100.00	100.00
2.2225	100.00	100.00	100.00
1.5875	100.00	100.00	100.00
1.27	100.00	100.00	100.00
0.9525	87.37	97.01	100.00
0.635	75.37	91.30	100.00
0.475	27.37	85.59	97.93
0.2	4.19	60.58	84.48
0.0425	2.38	32.23	3.95
0.018	1.99	16.53	1.32
0.0075	1.41	9.48	0.25

Table C-4. Weight Proportioning Calculations.

Item	Stockpile 1	RAP	Blend	Rubber
D_{50}	0.55	0.14	0.48	0.13
D_{MAXi}	0.22	0.06	0.19	----
Aggregate Wt (g)	4,443.00	2,384.98	6,827.98	----
Aggregate Vol (cm3)	1,689.35	924.82	2,614.17	----
Weight Proportion	0.651	0.349	1.000	----

interpolating between the two sizes that bracket the 50% passing size. This number is rounded to two decimal places in centimeters. Then the D_{MAXi} size is computed as $0.4D_{50}$ assuming the elongated particle shape factor (rounded to two decimal places in centimeters again). Once the decision has been made concerning how many of the available stockpiles will be used, it is a relatively simple process to compute the weight proportions for the blend. The process assumes that there are no significant interaction effects that would preclude usage of the air volume data from vibrating individual stockpile materials in the blend. The calculation sequence adheres to the following logic. Start with the largest stockpile (#1) aggregate weight, aggregate volume and air volume from Table C-2. Proceed down through the stockpiles as follows.

(a) From the vibration test, compute the fraction of the volume of the RAP stockpile that is aggregate (1,401.32/2,028.16=0.691).

(b) Completely fill the available volume in stockpile 1 (1,338.51cm^3) with the RAP. From step (a), only 69.1% of the available volume will be filled with aggregate which results in an aggregate volume and weight of RAP material of 924.82 cm^3 and 2,384.98 g, respectively (the small difference in hand computed values from the table is the result of more significant figures used in the spreadsheet program used to compute these values).

(c) Any additional stockpiles can be used to fill the remaining free volume by repeating steps (a) and (b). In this case, no other stockpiles are necessary.

(d) Total the aggregate weight (6,827.98).

(e) Compute the stockpile proportions (*e.g.*, 2,384.98/6,827.98=0.349). This is the blend recommended to ensure stone-on-stone contact with the stone skeleton created by the virgin stockpile playing a significant role in the load carrying capacity of the material.

Once the blend has been established, the final plant mix gradation is obtained by simply proportioning the materials according to the laboratory sieve analysis and the proportions of each size from step (e) above and multiplying the result by the lab-to-plant adjustment ratio, R_{lp}. This has been done in the final gradation computation given in Table C-5. In addition, computations of $ln(d/D)$ and $ln(P/100)$ have been performed for use in regression analysis on a spreadsheet to compute n in Equation C-3 (if regression analysis

Table C-5. Data for Asphalt Content Determination.

Sieve Size (cm)	Plant Gradation (% Passing by weight)	$ln(d/D)$	$ln(P/100)$	Surface Area (cm^2)
3.81	100.00	0.00	0.00	0
3.175	100.00	0.00	0.00	0
2.54	100.00	0.00	0.00	0
2.2225	100.00	0.00	0.00	0
1.5875	100.00	0.00	0.00	0
1.27	100.00	0.00	0.00	0.165426
0.9525	90.79	-0.29	-0.10	0.218201
0.635	81.68	-0.69	-0.20	1.196791
0.475	48.36	-0.98	-0.73	1.150033
0.2	24.41	-1.85	-1.41	1.309653
0.0425	13.16	-3.40	-2.03	3.22482
0.018	7.27	-4.26	-2.62	3.771168
0.0075	4.35	-5.13	-3.14	13.48976

tools are not available, use 0.0015 cm for the film thickness as an approximation). Using the data shown in the table, and forcing the intercept to zero, linear regression on a spreadsheet results in a value of 0.62 for n. Interpolating for film thickness between n=.45 (film thickness assumed to be 0.0007 cm), and n=0.9 (assumed film thickness 0.0030 cm) results in a film thickness of 0.0016 cm for this gradation. With regard to film thickness, it is noted that film thickness varies with the size particle, so the computed value is a crude approximation of the average for the blend. Also, thicker films may be used as rubber content increases. Concerning the role of fines, it has been suggested by other authors (*e.g., Tunnicliff 1962*) that fine particles with dimensions smaller than the film thickness often act as extenders of the fluid binder.

To the surface area of the aggregate, the surface area of the rubber must be added. By using the percent rubber in the RAP asphalt cement, the percent asphalt in the RAP, and the estimated surface area of 150 cm^2/g, the surface area of the rubber is found to be 0.576. The total surface area of the blended material then becomes 25.1 cm^2/g, which results in an asphalt content of 4.02% by weight of the aggregate using the equation

$$AC\% = 100\ (SA)\ (T_F)\ (\gamma_a) \tag{C-4}$$

where $AC\%$ is the percent asphalt by weight of the aggregate, SA is the total surface area per gram, T_F is the film thickness in cm, and γ_a is the unit weight of asphalt (assumed to be 1.03 g/cm^3 in this example). To this quantity, one must add a correction for absorption which is accomplished by adding the absorptions for each stockpile. This is approximated by multiplying the weight fraction of the stockpile by the water absorption (*e.g.*, for stockpile #1, 1.91*0.651=1.24). Finally, the total asphalt necessary is 4.02+1.63=5.65%. The asphalt available from the RAP for this blend is 1.92%, which implies that the virgin asphalt cement required is 3.73%.

5. The final step in designing the blend is to blend the asphalts to obtain suitable binder properties. At this time, penetrations are used for selecting the asphalt blend.

(a) Compute the percent asphalt available from the RAP (1.92/5.65=34.0%). On Figure C-1, move up from the baseline (0% CRM RAP AC) to 34% and draw a horizontal line.[8]

(b) Prepare the following 4 blends for penetration at 25°C (110°F):

1. 0% rejuvenating agent, 66% virgin AC, 34% CRM RAP AC,
2. 0% virgin AC, 66% rejuvenating agent, 34% CRM RAP AC,
3. 20% rejuvenating agent, 46% virgin AC, 34% CRM RAP AC, and
4. 20% virgin AC, 46% rejuvenating agent, 34% CRM RAP AC.

These four blends were selected to give a reasonably complete view of the variation of the penetration of the blends along the CRM RAP AC = 34% line.

(c) At each point for which penetration measurements have been taken (i.e., at each vertex and the four points from step b), mark the point and label it with the appropriate value of $log_{10}Pen$. Select the combination that has the desired penetration (*e.g.*, a 60 Pen material would be 1.778 on the chart) by interpolating between the two nearest points.[9]

(d) In addition to the desired penetration, it is also recommended that the temperature sensitivity of the material be tested by conducting an additional penetration test using the selected blend at 35°C (160°F) and at 25°C (110°F). Compute Pfeiffer and van Doormaal's slope, A, as

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[8] The easiest way to understand this figure is to recognize that the scale for CRM RAP AC goes from 0 to 100 when going vertically from the baseline to the vertex. Each of the other two scales is interpreted in the same way, and this is easiest to see by simply rotating the page so that each legend is right side up in sequence.

[9] This technique essentially reduces to that used in ASTM D4887 if only two components are to be used. Although linear interpolation is used in this proposed procedure to find the final combination, it does not assume that the relationship between $log_{10}Pen$ and the percent recycling agent is linear as is done in the ASTM procedure with $log_{10}Vis$. It has been observed that, in many cases, the lines are not necessarily straight as implied by the ASTM procedure. The additional 5 penetration measurements should help refine the interpolation process, especially if the user elects to plot contour lines using all 7 data points. Of course, in the two component mix problem, a maximum of four penetrations would be conducted with only three of the four actually being used in the final analysis (*e.g.*, the two ends of the rejuvenating agent = 0% baseline and the intersection of that line with the 34% CRM RAP AC line).

$$A = \frac{\log_{10}Pen_{35} - \log_{10}Pen_{25}}{35-25} \quad \text{(C-5)}$$

and the penetration index, *PI*, as

$$PI = \frac{20-500A}{1+50A} \quad \text{(C-6)}$$

Most unmodified asphalts have a PI between +1 and -1, with a value of below -2 indicating potential temperature susceptibility problems.

6. Perform an initial check on the effective utilization of the largest stockpile of material (stockpile 1) and the compaction effort by computing the voids in the coarse aggregate stockpile material in the compacted asphalt mixture. This computation is simply the bulk specific gravity, G_{bs}, of the compacted mixture times the weight fraction of the aggregate in the total mix times the weight fraction of the stockpile #1 material. The result of this calculation is the unit weight of the material from stockpile #1 in the total mix; this number should be greater than or equal to the unit weight of this material computed from Table C-2 (4,443/3,027.65=1.467g/cm^3).

7. Perform the mixture analysis test(s) desired. Either or both of the two tests are recommended for performance analysis: (1) the axial creep test (Tex-231-F), or (2) the repeated shear test. If axial creep is used, it is recommended (optional) that radial strains be measured and/or multiple confining pressures be used so that stone-on-stone contact can be reevaluated along with the analyses for long term performance. If repeated shear testing is performed, it should be done at constant height, and the axial load required to keep the height constant should also be recorded for the stone-on-stone contact analysis. Poisson's ratio is expected to equal or exceed 0.5 for mixes with good stone-on-stone contact. Materials with good stone-on-stone contact will have higher axial loads to maintain height in the shear test. The previous two statements should only be interpreted as identifying superior mixtures if the other portion of the analysis indicates that long term performance

is acceptable (i.e., impending failure can also be accompanied by high Poisson's ratios and high axial loads in the shear test under certain circumstances).

8. If the analysis procedure indicates that the mixture will not perform adequately, the following suggestions are offered for correction of the problem. Remove any stockpile that has the same D_{50} size as the D_{MAXi} size of the next larger stockpile, change the compaction effort to increase the unit weight of the mix (within the limits of what available compaction equipment will realistically be able to accomplish at the site, which will also be affected by characteristics of the platform against which the material will be compacted), and modify the asphalt with materials such as fillers or polymer modifiers (which may not cost effectively enhance resistance to rutting). If the problem is due to breakdown of the aggregate during mixing and/or compaction, a different parent material source for the aggregate must be selected. The performance evaluation does not evaluate moisture effects. This can be done through surface chemistry related tests and/or water permeability tests with leachate analysis.

9. If moisture effects are not measured, pick the highest asphalt content that gives the desired indication of long term performance from step 7 (minus the expected variability of asphalt content at the plant). If moisture effects are measured, select the lowest asphalt content that meets both moisture and performance requirements (plus the plant variability).

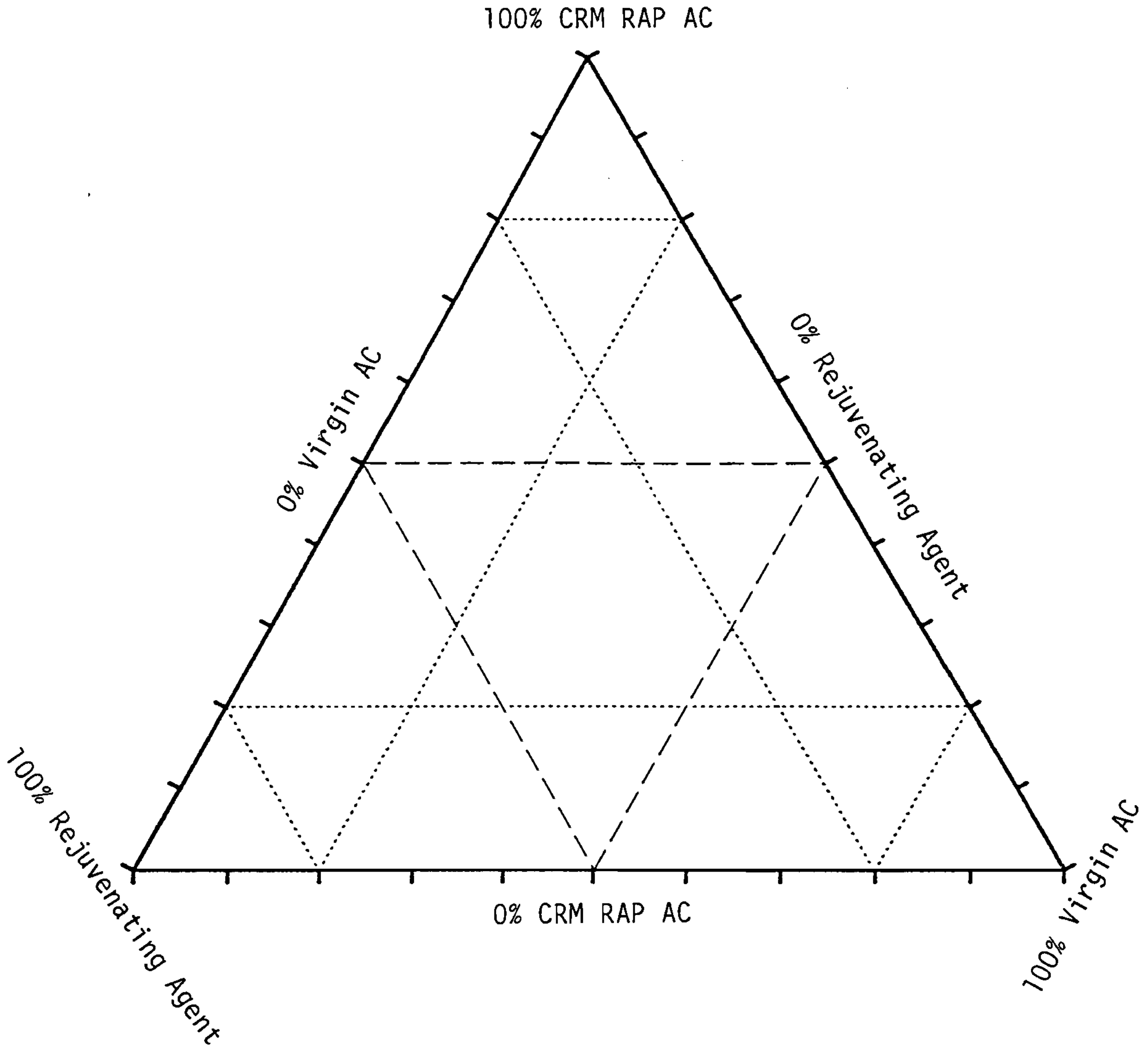

Figure C-1. Chart for Selecting Asphalt Blends.

APPENDIX D - ALTERNATIVE USES OF RECYCLED RUBBER

To address the problem of waste tire disposal to satisfactory utilization levels, alternative use of waste rubber should be under constant evaluation. The uses of tire rubber can be divided into three broad categories: energy conversion, whole tire applications, and shredded rubber applications. Suggestion of the following alternative uses is based on experiences reported by state agencies across the U.S. which have been documented by Epps (1994). The State of Minnesota estimates that it will exhaust its supply of waste tires within a few years primarily because of high volume use as fuel and as lightweight fill.

ENERGY CONVERSION

Tires are currently being burned as a fuel source. Tire Derived Fuel (TDF) is either produced in a facility solely dedicated to tires-to-energy operations, or is more directly used in electric power generation, production of cement in cement kilns, and for paper milling.

WHOLE TIRE APPLICATIONS

Retaining Walls

Tires are placed in parallel rows and backfilled with permeable material and covered behind, under, and over the tires with engineering fabric to prevent soil erosion. A thick layer of native soil is placed on top of the last layer of engineering fabric to encourage vegetation.

Impact Attenuators

Vehicle impact attenuators can be constructed using tires anchored together and filled with sand.

Safety Hardware

Whole tires can be used as bases for tubular traffic control markers and bases for vertical panel supports.

Fills and Embankments

Whole tires can be laid in a single or double layer over an existing roadbed as fill material in areas where the naturally occuring soil is peat or other soft soil.

Windbreaks

To reduce blowing sand and wind damage to young seedling trees in desert areas, woven tire walls and mats of tires can be constructed.

Culverts

Culverts can be built from tires bound together with steel reinforcing bars.

SHREDDED RUBBER APPLICATIONS

Lightweight Fill and Embankment

Tire chips are placed on excavated soil and compacted. A filter fabric is placed on the chips followed by a depth of earth fill.

Drainage Layer

Existing base material can be replaced with rubber chips laid in thicknesses of several centimeters covered by a layer of gravel. The tire chips can prevent capillary rise of ground water and thus minimize base contamination from the underlying subgrade.

Membranes

Membranes have been made with asphalt rubber binders and used for pond liners and to control moisture content in swelling clay soil subgrades.

Miscellaneous

Playgrounds, sidewalks, horse stall bedding, and potting soil amendments are some additional uses of the material. It is being considered for structural components and signs. Noise attenuating barrier walls can be constructed in high noise areas using the rubber as the attenuating medium in the structure.