RMRC Research Project No. 15
FINAL REPORT

DETERMINATION OF N_{DESIGN} FOR CIR MIXTURES USING THE SUPERPAVE GYRATORY COMPACTOR

by

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ABSTRACT

Cold in-place recycling (CIR) is a viable pavement rehabilitation technique that recycles 100% of the reclaimed asphalt pavement (RAP) in place, without the addition of heat. One of the barriers to wider use of CIR has been the lack of a suitable mixture design procedure. Researchers have shown that Superpave mix design technology is applicable to CIR mixtures if the N_{design} number of compaction gyrations can be established for the Superpave gyratory compactor (SGC).

The objective of this project was to determine the mix design compactive effort (N_{design}) using the SGC required to match the field density of CIR mixtures. RAP from seven CIR projects was obtained, along with the emulsified asphalt cement from each project. Samples were compacted using the SGC with the mix water and emulsion content from the field. Samples were compacted immediately after mixing and after a 30, 60 and 120 minute initial cure time. The change in density with compaction gyrations was monitored, and the N_{design} number of gyrations required to match the field density was determined.

The effect of initial cure time and RAP physical properties, such as gradation, percent flat and elongated particles, aggregate gradation and angularity on N_{design}, was evaluated. RAP shape, as measured by percent flaky pieces, was found to influence compacted field density. The N_{design} compactive effort for CIR mix design also was established.

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INTRODUCTION

INTRODUCTION

There are currently many tons of asphalt pavements milled each year by various state and local highway agencies. The majority of the millings are not recycled, but are disposed of in landfills. These asphalt millings, or reclaimed asphalt pavement (RAP), contain reusable natural resources of asphalt cement and mineral aggregates. Cold in-place recycling (CIR) is a viable pavement rehabilitation technique that recycles 100 percent of the RAP in place, resulting in no valuable resources being buried in landfills and considerable savings of dollars and energy.

Researchers at the University of Rhode Island (URI) have shown that the Superpave gyratory compactor (SGC) can be used to determine optimum moisture and asphalt emulsion content of CIR mixtures.(1) The URI study evaluated a volumetric mix design procedure for CIR mixtures using two different compactors, a Marshall compaction hammer and the SGC. Compaction using the Marshall hammer was based on the work of Task Force 38, *Report on Cold Recycling of Asphalt Pavements*,(2) which recommended 50-blow compaction. The URI study recommended the use of the SGC; however, the number of compaction revolutions was not definitively established and needs to be established based on the achievable field density for CIR pavements.

One of the fundamental principles behind a volumetric mix design is the selection of an appropriate compactive effort. The mix design compactive effort must produce a test sample with void properties similar to those that the same mix would experience in the field. Historically, 50-blow Marshall compactive effort has been used by a majority of highway agencies for CIR mixture design.(2) A recent study by Cross has shown that compaction to 75 blows per side at 43.3°C (110°F) was necessary to replicate the field density of a CIR mixture in Kansas.(3)

Others have evaluated the use of the SGC for cold mix design as well. Lauter and Corbett [4] evaluated the N_{design} number of gyrations required to reproduce field densities for CIR mixtures in Ottawa-Carleton, Canada. The authors reported a wide range in required N_{design} gyrations and that a single compactive effort could not be established. Mallick [5, 6] has recommended 50 and 75 gyrations be used for mix design of full depth reclamation mixes. However, his recommendations were based on only one RAP gradation. The effect of cure time on CIR mix design compactive effort has not been fully established either.

Preliminary mix design results using the SGC and the Marshall hammer are available from the URI study for two mixes, one in Kansas and one in Ontario.[1] The Marshall mix design was a 50-blow mix design and the Superpave mix design used 50 gyrations. The differences in density between SGC and Marshall compacted samples were approximately 160 kg/m^3 (10 pcf). This relates to a difference in voids total mix (VTM) of approximately six percent. The SGC samples had a compacted density much higher than typically encountered in the field. Laboratory compacted density is often used during construction quality control testing as a target value to ensure adequate field compaction with 95 percent of the target value typically required. The use of inappropriate mix design compactive effort (N_{design}) can result in target values being set unrealistically high, thus making contractor compliance difficult using conventional compaction techniques. Setting unreasonable compaction target values could result in numerous failed test results from otherwise acceptable CIR pavements.

OBJECTIVE

The main objective of this study was to determine the N_{design} number of compaction revolutions for CIR mix design required to duplicate the field density of CIR mixtures. A second objective was to evaluate the effect of initial cure time on mix design compactive effort.

SCOPE

RAP from seven projects was obtained, along with the emulsified asphalt cement used on the project. Samples were compacted using the SGC with the mix water and emulsion content obtained from the field test sites. Samples were compacted immediately after mixing and after a 30, 60 and 120 minute initial cure time. The change in bulk specific gravity with compaction gyrations was monitored. The N_{design} number of gyrations required to match field density was determined by comparing SGC compacted density to the field compacted density. The effect of initial cure time on the N_{design} number of gyrations was evaluated as well.

Chapter 2
FIELD OBSERVATIONS

GENERAL OBSERVATIONS

The pavements selected for evaluation were chosen to provide as wide a variety of materials, climates and traffic as practical. The pavements were selected from the regions that currently utilize CIR. The majority of the sites were selected from the central and northeast sections of the United States. Figure 1 indicates the general location of the test sites. Table 1 gives the location and project number of each site and table 2 is a summary of available traffic data.

The majority of the test sites were located on level tangents of two-lane rural highways. All of the pavements, except site 5, carried low volumes of traffic. The CIR at each test site consisted of recycling 100-mm (4-inch) deep except for sites 4 and 5. Site 4 was recycled 115-mm (4.5-inch) deep and site 5 was recycled 75-mm (3-inch) deep. The wearing surface at all sites was a new HMA overlay, which varied in thickness from 38 to 115 mm (1.5 to 4.5 inches). The depth of the remaining pavement beneath the CIR layer varied considerably from site to site, and the information was not available at some of the sites.

SITE DESCRIPTIONS

Site 1

Site 1 was located on US-283 in Ford County, Kansas. The recycling project was a Kansas DOT experimental project, Project No. 106-283 K 6354-01, comparing the use of type C fly ash to asphalt emulsion with lime slurry as additives. The one-way traffic at the test site was 140 80 kN (18 kip) equivalent single axle loads (ESALs) per day, with an average annual daily traffic (AADT) of 1033 vehicles and 21.5 percent trucks. This site

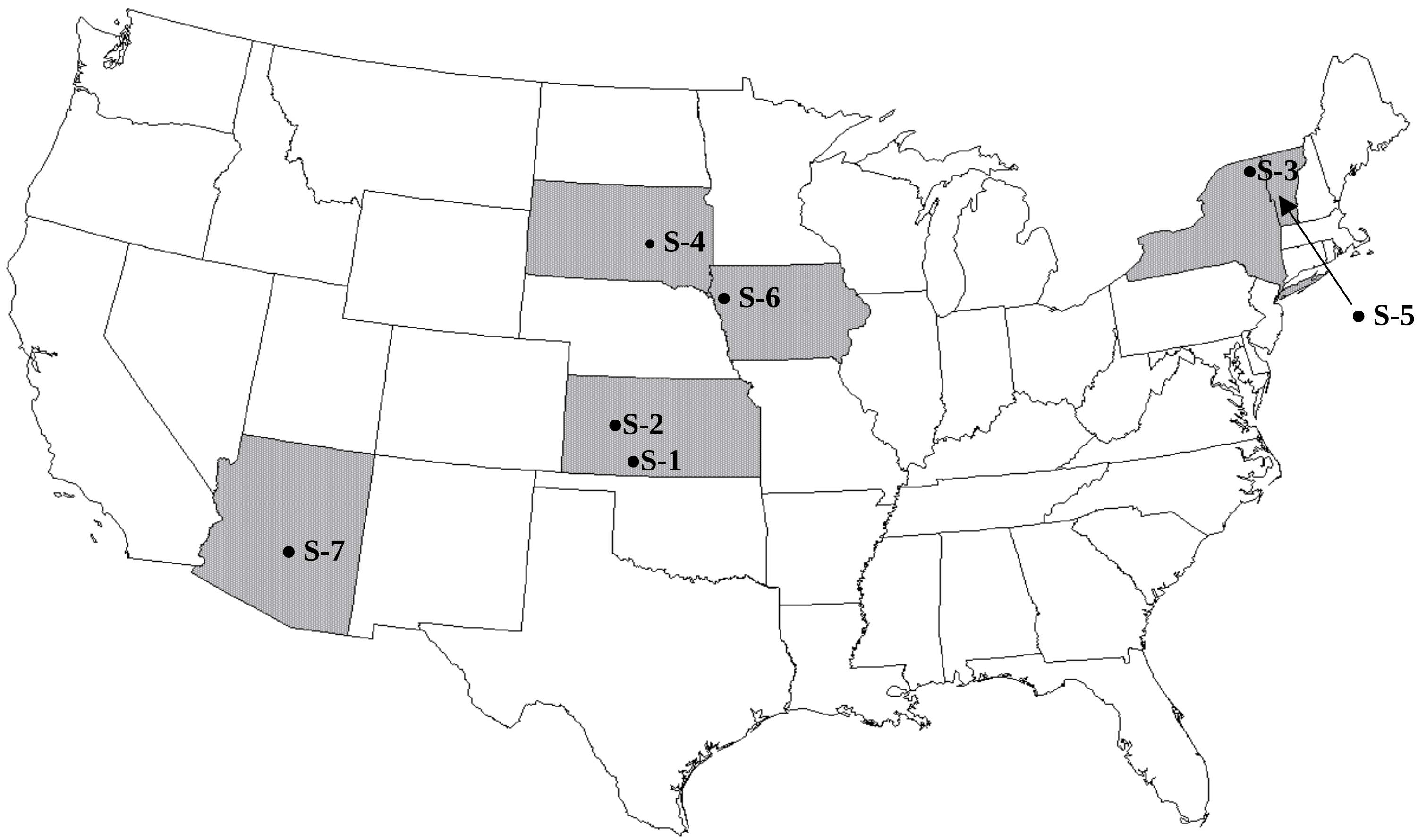

Figure 1. Location of Test Sites

Table 1. Test Site Location and Project Number

Site	State	Route	County	Project No.	Lane
1	Kansas	US-283	Ford	106-283 K 6354-01	Northbound
2	Kansas	US-24	Graham	106-24 K 7797-01	Westbound
3	New York	US-11	Franklin	N/A	Eastbound
4	South Dakota	US-281	Jerauld	P 0281(56)95	Southbound
5	Vermont	Rt.-2	Montpelier	NH 9808 (1) 5	N/A
6	Iowa	K-42	Plymouth	FM-C075(71)-55-75	Northbound
7*	Arizona	N/A	Maricopa	N/A	N/A

*Sampled by others N/A = Information not available.

Table 2. Available Traffic Data.

Site	State	Route	One -Way Traffic AADT	% Trucks	ESALs
1	KS	US-283	1033	21.5	140
2	KS	US-24	640	16.8	55
3	NY	US-11	2985	13.0	N/A
4	SD	US-281	1150	19.1	N/A
5	VT	Rt-2	7100	N/A	440
6	IA	K-42	440	12.0	N/A
7*	AZ	N/A	N/A	N/A	N/A

*Sampled by others N/A = Information not available.

was not sampled as a part of this study, but sufficient material (RAP) was available from a previous study [7] by the author to allow incorporation of this site into the study.

The exact emulsified asphalt cement (EAC) used on the project was not available. The EAC from site 2 was the same grade, CSS-1, and was from the same supplier. Therefore, the CSS-1 from site 2 was used with the RAP from Site 1. The EAC content was 1.5 percent and hydrated lime, at a rate of 1.5 percent, was added at the cutting head by injecting 4.5 percent slaked quicklime slurry, both by weight of the RAP. The RAP was sampled without the slurry or the EAC.

The CIR was placed in 1997, and the pavement consisted of approximately 38 mm (1.5 inches) of HMA as a wearing surface and 100 mm (4 inches) of CIR over an undetermined amount of old pavement. Figure 2 shows the CIR section at site 1.

Site 2

Site 2 was located east of Hill City on US-24 in Graham County, Kansas. The recycling project was a Kansas DOT project, Project No. 24-33 K 7536-01. The one-way traffic at the test site was 55 ESALs per day with an AADT of 640 and 16.8 percent trucks. The recycling consisted of a new 40-mm (1.5-inch) HMA overlay over a 100-mm (4-inch) CIR mix. Samples of RAP were obtained on May 4, 2000, from the westbound lane. The EAC was obtained from the supplier. Figure 3 shows the compaction of the CIR mix at site 2.

The recycling was accomplished using a recycling train that consisted of a milling machine operating in an upcutting mode, a screening and crushing unit, and a pugmill. The RAP was screened to produce 100 percent passing a 31.5-mm (1.25-inch) screen. The EAC was a CSS-1 applied at a rate of 2.15 percent by weight of the RAP. Hydrated lime at a rate of 1.6 percent was added at the cutting head by injecting 4.2 percent slaked quicklime slurry, both by weight of the RAP. The RAP was sampled with the slurry included but without the EAC.

Site 3

Site 3 was located approximately two miles west of Chateaugay on US-11 in Franklin County, New York. The recycling project was a New York DOT project. The one-way

Figure 2. CIR Process, Site 1, US-283

Figure 3. CIR Compaction, Site 2, US-24

AADT at the test site was 2,985 vehicles per day with 13 percent trucks. The recycling consisted of a new 40-mm (1.5-inch) HMA overlay over a 100-mm (4-inch) CIR mix.

Add-stone was incorporated in the CIR mixture, at a rate of 18 percent by weight of RAP, by placing the stone on the pavement in front of the recycling train. Samples of RAP, with the add-stone included, were obtained on June 15, 2000, from the eastbound lane. The EAC was obtained from the supplier. Figure 4 shows the add-stone in front of the recycling train at site 3.

Figure 4. CIR Process With Add-Stone, Site 3, US-11

Recycling was accomplished using a recycling train that consisted of a milling machine, operating in a down-cutting mode at 135 rpm, a screening and crushing unit, and a pugmill. The 135-rpm speed of the cutting head on the milling machine was faster than the typical speed of 90 rpm. The RAP was screened to produce 100 percent passing a 38.1-mm (1.5-inch) screen. The EAC was an HFMS-2 applied at a rate of 2.06 percent by weight of the RAP plus add-stone. Water, applied at the cutting head, was introduced to the mix at a rate of 2.0 percent by weight of the RAP plus add-stone. The RAP was sampled with the add-stone included but without the EAC.

Site 4

Site 4 was located on US-281 in Jerauld County, South Dakota from the junction with South Dakota Highway 34 north to the Beadle County line. The recycling project was a South Dakota DOT project, Project No. P 0281(56) 95. The AADT at the test site was 1,150 vehicles per day with 19.1 percent trucks. The recycling consisted of milling 115-mm (4.5 inches) deep over a 4.4-m (14.5-foot) width per lane and placing the CIR 5.8 m (19 feet) wide. A new 63.5-mm (2.5-inch) HMA overlay was placed over the CIR mixture. Samples of RAP were obtained on July 8, 2000, from the southbound lane. The EAC was sampled by the DOT and supplied by the contractor.

Recycling was accomplished using a recycling train similar to that used on site 2. The RAP was screened to produce 100% passing a 31.5mm (1.25-inch) screen. The EAC was a high float AE200S applied at a rate of 1.1 percent by weight of the RAP. Water was applied at a rate of 3.0 percent by weight of RAP.

Site 5

Site 5 was located on Route 2 near Montpelier, Vermont. The recycling project was a Vermont DOT project, Project No. NH 9808 (1) 5. The 20-year design ESALs for the project were 4.5 million with an estimated current daily ESALs of 440. Current AADT along the entire project ranged from 7,100 to 11,700 vehicles per day, with 7,100 vehicles in the test section. The recycling consisted of a new 45-mm (1.75-inch) HMA surface mix with a 70-mm (2.75-inch) HMA binder mix overlay over a 75-mm (3-inch) CIR mix.

The project was recycled in July 2000. RAP without EAC was obtained from the project and the EAC was obtained from the supplier. Recycling was accomplished using the same recycling train and compaction equipment as used on site 3.

Site 6

Site 6 was located on County Road K-42 in Plymouth County, Iowa, from Bruinsville north to county road C-12. The recycling project was an Iowa DOT project, Project No. FM-C075(71)-55-75. The estimated AADT over the project ranged from 350 to 440 vehicles per day. The Iowa DOT uses an estimate of 12 percent trucks for county roads. The recycling consisted of milling 100 mm (4.0 inches) deep over a 3.5-m (11.5-foot) width per lane. A new 75-mm (3.0-inch) HMA overlay was placed over the CIR mixture. Samples of RAP were obtained on September 15, 2000, from the northbound lane. The contractor supplied the EAC.

Recycling was accomplished using a recycling train similar to that used on sites 1, 2 and 4. The RAP was screened to produce 100 percent passing a 31.5-mm (1.25-inch) screen. The EAC was a high float HFE-300 applied at a rate of 2.0 percent by weight of the RAP. Water was applied at a rate of 1.5 percent by weight of RAP.

Site 7

Site 7 was a Bureau of Indian Affairs project near Sacaton, Arizona. The project was sampled as a part of Lee's work at URI [(1)] and samples were provided to the University of Kansas. This project used a recycling agent rather than an asphalt emulsion. The recycling agent was Cyclogene HE, applied at a rate of 2.5 percent by weight of the RAP. Water was applied at a rate 2.0 percent by weight of the RAP. Little reliable compaction and in-place density information was available for this site. The RAP from site 7 was evaluated in the laboratory, but was excluded from some of the analysis due to the lack of reliable field information.

Chapter 3
LABORATORY TEST PROCEDURES

RAP

Contractor personnel obtained samples of RAP for sites 2-7. All samples of RAP were obtained without the emulsified asphalt cement (EAC). Approximately 100-150 kg (220-330 lbs.) of RAP was obtained from each site. The RAP samples for site 1 came from a previous project by the author.(7) For the remaining sites, the RAP was generally sampled off the conveyor belt from the crushing screening unit prior to entering the pugmill. The CIR mix from site 2 contained slaked lime slurry, which was added at the cutting head. The RAP from this site was obtained from the windrow deposited by the pugmill. EAC was not introduced into the pugmill where the RAP sample was obtained.

The gradation of the RAP, as received, was determined in general accordance with AASHTO T 27. Approximately half of the RAP from each site was placed in large flat pans and placed in a forced draft oven at 60^{o}C (140^{o}F) for 24 hours to remove surface moisture. The material was then sieved over a 38.1-mm (1.5-inch) sieve through 2.36-mm (No. 8) sieve, inclusive, and the material separated into sizes for batching. The gradation was determined and if the percentage of material passing the 2.36-mm (No. 8) sieve exceeded 25% of the total, the material passing the 2.36-mm (No. 8) sieve was sieved over the 1.18-mm (No. 16) sieve. This material was separated into sizes, and the gradation recalculated. To determine the gradation of the RAP through the 0.075-mm (No. 200) sieve, two 1,000-g samples of the material retained in the pan (passing 2.36 mm or 1.18 mm) were sieved over the 2.38-mm (No. 8) sieve through the 0.075-mm (No. 200) sieve, inclusive. The gradation of the entire RAP was then calculated.

From the complete gradation of the RAP, the fineness modulus was determined in accordance with AASHTO T 27. The fineness modulus is a parameter used to evaluate sands for use in Portland cement concrete and does not include the material passing the

0.150-mm (No. 100) sieve. HMA mixtures can contain 10-20 percent passing the 0.015-mm (No. 100) sieve; therefore, Hudson's A coefficient was determined. Hudson's A coefficient is very similar to the fineness modulus except it uses percent passing, rather than percent retained, and uses the 0.075-mm (No. 200) sieve. Hudson's A coefficient has been shown to better quantify bituminous mixture performance than the fineness modulus.(8) The surface area of the RAP was determined using Hveem's surface area factors as presented in the Asphalt Institute's *MS-2, Mix Design Methods for Asphalt Concrete.*(9) RAP has very little material passing the 0.300-mm (No. 50) sieve; therefore, the gradations have very little surface area when compared to conventional HMA mixtures.

RAP millings tend to be flaky in shape. To quantify the flakiness of the RAP, the percent flaky particles were determined by comparing the largest dimension to the smallest dimension, in general accordance with ASTM D 4791. This is neither a flat nor elongated particle as described by ASTM in their test method D 4791. Elongated particles are defined as the ratio of length to width and flat particles as the ratio of width to thickness. Flakiness, for the purpose of this study, is defined as the ratio of the length to thickness or largest dimension to smallest dimension.

Two 2,000-g samples of the RAP from each site were batched to the appropriate gradation and the physical properties determined. The theoretical maximum density (Gmm) was determined in accordance with AASHTO T 209. Next, the asphalt content was determined using the ignition furnace in accordance with AASHTO T 308. The gradation of the recovered aggregate was determined in accordance with AASHTO T 30. The crushed face count of the recovered coarse aggregate was determined in accordance with ASTM D 5821. The fine aggregate angularity of the recovered aggregate was determined in accordance with Kansas DOT Test Method KT-50.(10) KT-50 is similar to AASHTO T 304, except the volume of the aggregate is measured directly using a 200-ml flask rather than indirectly using the bulk specific gravity. The surface area, fineness modulus and Hudson's A coefficient were determined for the extracted aggregate in the same manner as with the RAP.

MIXING, COMPACTION, AND CURING OF LABORATORY SAMPLES

Mixing

All samples were mixed in general accordance with the recommendations of Lee.[1] Samples of RAP were batched to 4,000 grams. The appropriate amount of water was added to the RAP and mixed for 30 seconds. The EAC was added and the material was mixed for an additional 90 seconds. All samples were mixed using a mechanical mixer. The RAP, compaction molds and mix water were at room temperatures, approximately 25^{o}C (77^{o}F). The EAC was heated to 65^{o}C (150^{o} F). EAC and mixing water contents were those used in the field.

Initial Curing

Typical laboratory compaction procedure for cold mixes entails compacting samples after the EAC breaks. However, not everyone does this and some owner/agencies compact samples immediately after mixing. One indication of the breaking of an EAC is a change in color from brown to black. In the field, CIR mixtures are usually compacted just when the outside of the windrow turns from brown to black. However, the majority of the EAC in the CIR mixture in the windrow has not broken. In the laboratory it was difficult to determine when the EAC broke. This was due, in part, to the low emulsion contents, less than 3.0 percent, the black color of the RAP, and fluorescent lighting. To overcome this, samples were compacted immediately after mixing and 30, 60 and 120 minutes after mixing. The samples were placed in a flat pan to cure and/or break for the allotted time.

Compaction and Final Curing

After the initial cure time, the samples were compacted using the Superpave Gyratory Compactor (SGC). Samples were compacted to 50 gyrations in accordance with the proposed method outlined by Lee [1] and AASHTO TP 4. Samples were compacted at ambient temperatures, 25°C (77°F). The height of the samples were monitored and recorded continuously during compaction. After compaction, the samples were extruded from the compaction mold and placed in a forced draft oven at 60°C (140°F) for 48 hours for final curing.

LABORATORY TESTING

Bulk Specific Gravity

After the 48-hour oven cure, the samples were removed from the oven and allowed to cool to room temperature. Next, the bulk specific gravity was determined in accordance with AASHTO T 166. Based on the recorded heights and final bulk specific gravity, the bulk specific gravity with each compaction revolution was calculated as specified in AASHTO TP 4.

It was anticipated that some of the samples would have high air void contents, well in excess of eight percent. Previous research on HMA has shown that Marshall compacted samples with air voids over eight percent had significantly different bulk specific gravities when determined using AASHTO T 166 and when using parafilm coated samples.[11] Therefore, some of the samples were tested for bulk specific gravity using the CoreLok™ procedure,[12] a proposed replacement for AASHTO T 275. The test was performed in accordance with the manufacturer's recommendations.[12]

Permanent Deformation

Two SGC compacted samples from each of the four curing conditions were tested for resistance to permanent deformation using an Asphalt Pavement Analyzer (APA) in the dry mode. The test was performed in accordance with the manufacturer's recommendations.[13] The test temperature was selected to represent approximately the 85 percent reliability of the maximum anticipated pavement temperature for each mix. The pavement mix temperature was determined using the formula developed from the LTPP database in LTPPBind version 2.1.[14] The test temperature was reduced one PG grade to account for the presence of a surface mix over the CIR layer. The APA test parameters were as follows:

Hose Pressure:	690 kN/m^2 (100 psi)
Wheel Load:	0.44 kN (100 lbs.)
Total Cycles:	8,000
Sample Preconditioning:	6 hours in air at test temperature.

Indirect Tensile Strength

The samples that were not tested in the APA were tested for indirect tensile strength in accordance with ASTM D 4123.

Chapter 4
TEST RESULTS

AGENCY REPORTED COMPACTION

Typical Compaction Procedures

All of the sites were recycled using a recycling train as previously described in chapter 2. Compaction of the CIR layer was accomplished using heavy pneumatic rollers and static and vibratory steel wheel rollers. Typical compaction procedures consisted of initial rolling with a 11.4 Mg (12.5 ton) static steel wheel roller followed by eight to 10 passes with a pneumatic roller. Pneumatic rollers were typically seven-tired rollers weighted to 27.3 Mg (30 tons) with 620 kPa (90 psi) tire pressure. Finish rolling consisted of two to three vibratory passes of the 11.4 Mg (12.5 ton) vibratory steel wheel roller followed by the same number of passes with the same roller in the static mode. Figure 3 showed typical CIR compaction equipment.

Reported Agency Compaction Results

Table 3 shows the agency methodology used to determine the target value for compaction control and the target value for each site, if available. Agencies typically used nuclear moisture-density meters and field moisture content samples to monitor percent compaction. The results in table 3 indicate that the projects met the minimum percent compaction requirements. A brief description of the agency reported compaction control procedures and results are provided below.

Site 1

The target density for this CIR mixture was determined by the Kansas DOT using field produced samples. Compaction control was a minimum of 95% of a 50-blow Marshall compacted sample. Compaction was reported as exceeding minimum requirements.

Table 3. Agency Target Value and Reported Percent Compaction

Site	State	Route	Agency Method Used to Determine Compaction Target Value	Target Value	Reported Percent Compaction
1	KS	US-283	Minimum 95% of 50-Blow Marshall of Laboratory Compacted Field Sample	N/A	> 95%
2	KS	US-24	Minimum 97% of Field Test Strip	1956 kg/m^3	99 - 104 %
3	NY	US-11	No Requirement	N/A	N/A
4	SD	US-281	Minimum 97% of Field Test Strip	N/A	> 97%
5	VT	RT 2	Minimum 95% of 50-Blow Marshall of Laboratory Compacted Field Sample	2035 kg/m^3	95 - 103 %
6	IA	K-42	Minimum 92% of 75-Blow Marshall of Laboratory Compacted Field Sample	1989 kg/m^3	94 - 98 %
7	AZ	N/A	N/A	N/A	N/A

N/A = Information not available.

Site 2

Site 2 was constructed three years later than site 1 and the target density for the CIR mixture was changed by the Kansas DOT to a percentage of a test strip density. The minimum specified percent compaction is 97 percent of the test strip density or target value. The target value for this project was 1956 kg/m^3. All density measurements exceeded the specified minimum percent compaction.

Site 3

This project was a New York DOT maintenance contract and density testing of the CIR was not required.

Site 4

The South Dakota DOT determines the target density for the CIR mixtures using a field test strip. The minimum specified percent compaction was 97 percent of the target value. All density measurements exceeded 97 percent of the target value.

Site 5

Vermont DOT specifications require a minimum compaction for CIR of 95 percent of a target density. The target density is determined from a 50-blow Marshall laboratory compacted sample. The target density was 2035.3 kg/m^3 (127.0 pcf) with a minimum field compacted unit weight of 1934.3 kg/m^3 (120.7 pcf). Compaction exceeded the required minimum.

Site 6

Target densities for county road projects in Iowa are a minimum of 92 percent of a laboratory compacted sample. A field sample is obtained each day and transported to an Iowa DOT district materials laboratory where it is immediately compacted to 75 blows per side with a Marshall hammer. The laboratory compacted dry density for September 15, 2000, was 1988.8 kg/m^3 (124.1 pcf), with a minimum compacted density of 1830.1 kg/m^3 (114.2 pcf). All density tests in the test section exceeded the minimum density requirement.

Site 7

The project was sampled as a part of Lee's work at URI [(1)] and samples were provided to the University of Kansas. Agency compaction requirements and results were not available.

FIELD TEST RESULTS

Sites 2-6 were sampled as a part of this research project and followed a field sampling and testing plan. Site 1 was previously sampled under a different project by the author.[(7)] Site 7 was a part of project by URI [(1)] and samples were supplied to the University of Kansas.

For sites 2-6, a 30-m long test section was generally laid out for sampling and testing. After placement and compaction, the in-place density was determined using a nuclear moisture-density meter. Wet density readings were obtained using backscatter or direct transmission modes at 50 or 100-mm (2-4 inch) depths. Figure 5 shows a typical test section with density testing.

In addition to density tests, samples of CIR mix were obtained for moisture determination. Two to three samples were obtained from the compacted roadway, sealed in plastic bags, and returned to a laboratory for moisture content determination. The moisture content was used to convert the wet density to a dry density. Table 4 shows the results of the field density testing for each site.

Figure 5. Field Density Determination

Table 4. Results From Field Nuclear Moisture-Density Gauge Testing

				Wet Density		Moisture	Dry Density		Temperature			
				Range	Average	Content	Average		RAP		Air	
Site	State	Route	Mode*	(kg/m^3)	(kg/m^3)	(%)	(kg/m^3)	(pcf)	(C)	(F)	(C)	(F)
1	KS	US-283	100 mm	N/A	N/A	N/A	2105.8	131.4	N/A		N/A	
2	KS	US-24	100 mm	1936 - 2043	1998.5	2.3	1953.5	121.9	31.7	89	23.9	75
3	NY	US-11	50 mm	1981 - 2125	2073.7	3.1	2011.2	125.5	30.0	86	20.0	68
4	SD	US-281	50 mm	1973 - 2058	2033.7	1.3	2008.0	125.3	43.3	110	33.3	92
5	VT	RT-2	Backscatter	N/A	N/A	N/A	2014.4	125.7	N/A		N/A	
6	IA	K-42	50 mm	1870 - 1950	1910.3	1.6	1879.8	117.3	26.1	79	15.0	59
7	AZ	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A		N/A	

* Either Backscatter or Direct Transmission with Depth Indicated

N/A = Information not available.

LABORATORY TEST RESULTS

RAP and Aggregate Properties

The gradation of the RAP, as received, was determined in accordance with AASHTO T 27. From the complete gradation of the RAP, the fineness modulus (FM) and Hudson's A coefficient were determined. The FM was determined in accordance with AASHTO T 27. The surface area of the RAP was determined using Hveem's surface area factors.[9] The percent flaky particles of the RAP were determined by comparing the largest dimension to the smallest dimension, in general accordance with ASTM D 4791. The properties of the RAP are shown in table 5.

Two 2,000-g samples of the RAP from each site were batched to the appropriate gradation and the physical properties determined. The theoretical maximum density (Gmm) was determined in accordance with AASHTO T 209. Next, the asphalt content was determined using the ignition furnace in accordance with AASHTO T 308. These properties are also shown in table 5.

The aggregate was recovered from the ignition furnace and the gradation determined in accordance with AASHTO T 30. The crushed face count of the recovered coarse aggregate was determined in accordance with ASTM D 5821. The fine aggregate angularity (FAA) of the recovered aggregate was determined in accordance with Kansas DOT Test Method KT-50.[10] The surface area, FM and Hudson's A coefficient was determined for the extracted aggregate in the same manner as with the RAP. The results are shown in table 6. Figures 6-12 are plots of the RAP and aggregate gradations.

Table 5. RAP Gradations and Physical Properties

State	KS	KS	NY*	SD	VT	IA	AZ
Route	US-283	US-24	US-11	US-281	RT-2	K-42	N/A
Sieve Size	Site						
	1	2	3	4	5	6	7
(mm)	Percent Passing						
38.1	100	100		100	100	100	100
25.4	98.6	97.9	100	99.4	99.5	92.3	97.5
19.0	95.7	93.2	95.6	95.1	93.9	81.2	94.4
12.5	81.7	83	78.8	80.4	77.5	61.2	78.1
9.5	69.0	75.1	67.6	68.6	62	50.7	64.8
4.75	41.5	54	37.7	31.9	34.2	31.6	36.2
2.36	22.9	35.2	19.2	15.0	19.2	20.8	18.6
1.18	11.1	18.3	9.5	6.7	11.2	12.7	10.1
0.600	5.0	8.5	4.3	2.8	5.2	5.3	5.1
0.300	1.6	2.7	1.7	1.1	2.2	1.9	2.3
0.150	0.4	0.7	0.5	0.4	0.8	0.7	0.8
0.075	0.0	0.2	0.1	0.1	0.2	0.2	0.2
AC (%)	5.70	6.52	5.13	8.19	6.38	7.52	6.68
Gmm	2.400	2.360	2.508	2.401	2.418	2.418	2.402
Flaky Particles (%)							
2:1	55.8	80.5	62.4	62.4	50.5	70.8	65.2
3:1	2.0	16.7	17.9	5.9	14.8	24.2	11.2
Surface Area (kg/m^3)	0.84	1.38	0.79	0.60	0.93	0.93	0.92
Hudsons A	2.47	2.88	2.36	2.22	2.29	2.05	2.33
FM	5.53	5.12	5.64	5.78	5.71	5.95	5.68

* Includes Add-Stone
N/A = Not Available

Table 6. Recovered Aggregate Gradation and Physical Properties

State	KS	KS	NY*	SD	VT	IA	AZ
Route	US-283	US-24	US-11	US-281	RT-2	K-42	N/A
Sieve	Site No.						
Size	1	2	3	4	5	6	7
(mm)	Percent Passing						
25.4			100				100
19.0	100		97.2	100	100	100	99.6
12.5	96.6	100	89.4	99.5	99.4	94.2	96.4
9.5	91.3	98.4	86.3	94.2	90.9	88.2	91.2
4.75	75.6	87.9	67.9	75.7	68.7	73.2	69.9
2.36	59.9	72.8	48.4	57.6	56.3	60.9	53.1
1.18	46.3	55.5	37.6	42.8	46.2	46.6	41.4
0.600	33.7	39.5	30.2	31.2	35.5	28.7	30.6
0.300	20.5	24.6	22.5	20.3	24.6	14.2	19.6
0.150	11.4	15.7	14.7	10.8	14.2	8.0	10.9
0.075	7.6	11.9	9.9	7.1	8.2	5.9	7.0
Crushed Faces (%)							
0	9.7	5.6	0.0	42.4	1.2	47.5	21.6
1	7.2	2.4	0.0	14.6	0.0	4.5	4.5
2 or More	83.1	92.0	100.0	43.0	98.8	48.1	73.9
FAA (%)	41.1	39.0	43.4	39.7	42.1	40.2	39.0
Surface Area (kg/m^3)	7.68	10.34	8.57	7.27	8.45	6.18	7.11
Hudsons A	4.46	4.06	4.15	4.40	4.45	4.26	4.23
FM	3.61	4.06	3.95	3.67	3.64	3.80	3.84

* Includes Add-Stone
N/A = Not Available

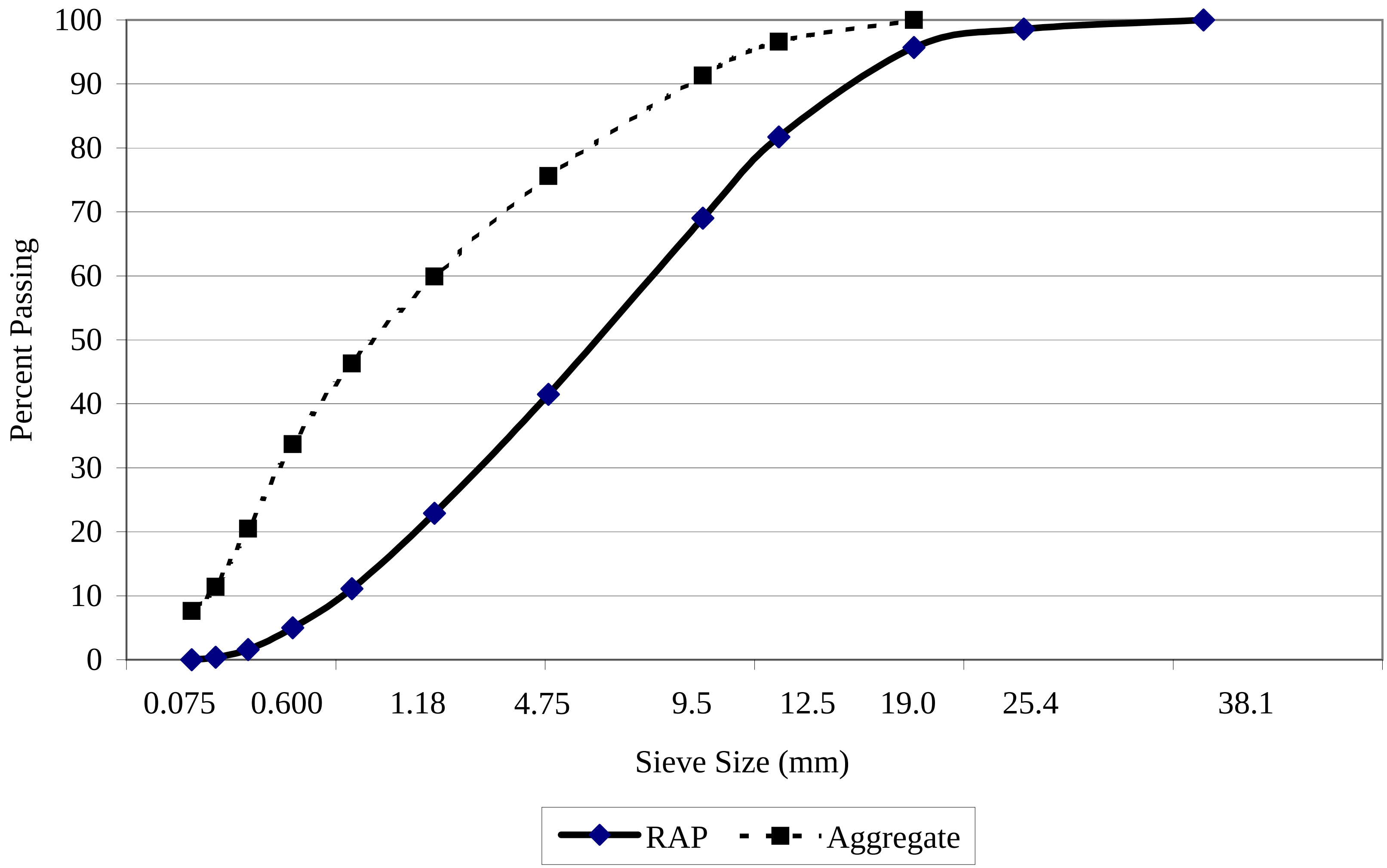

Figure 6. RAP and Extracted Aggregate Gradation, Site 1

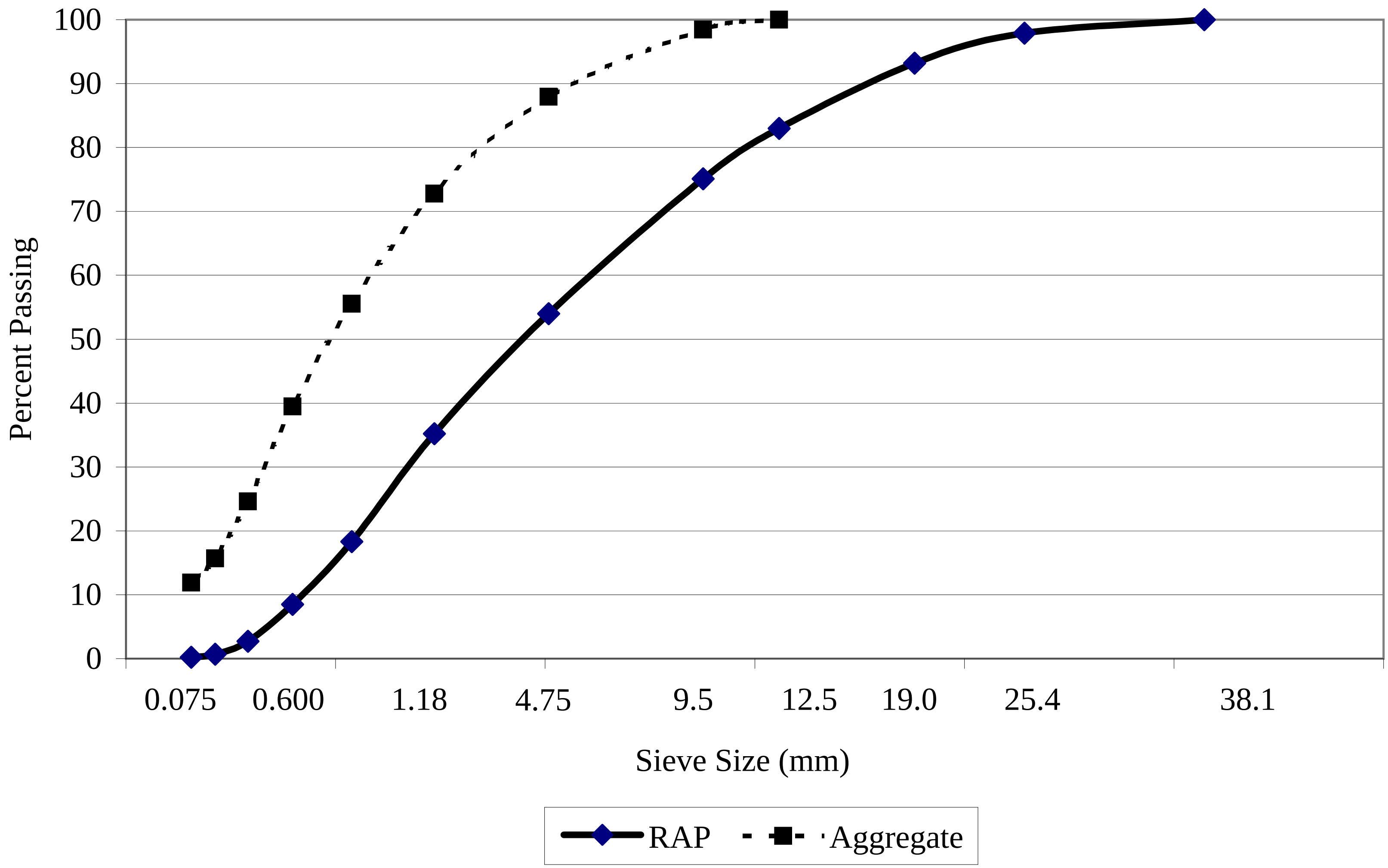

Figure 7. RAP and Extracted Aggregate Gradation, Site 2

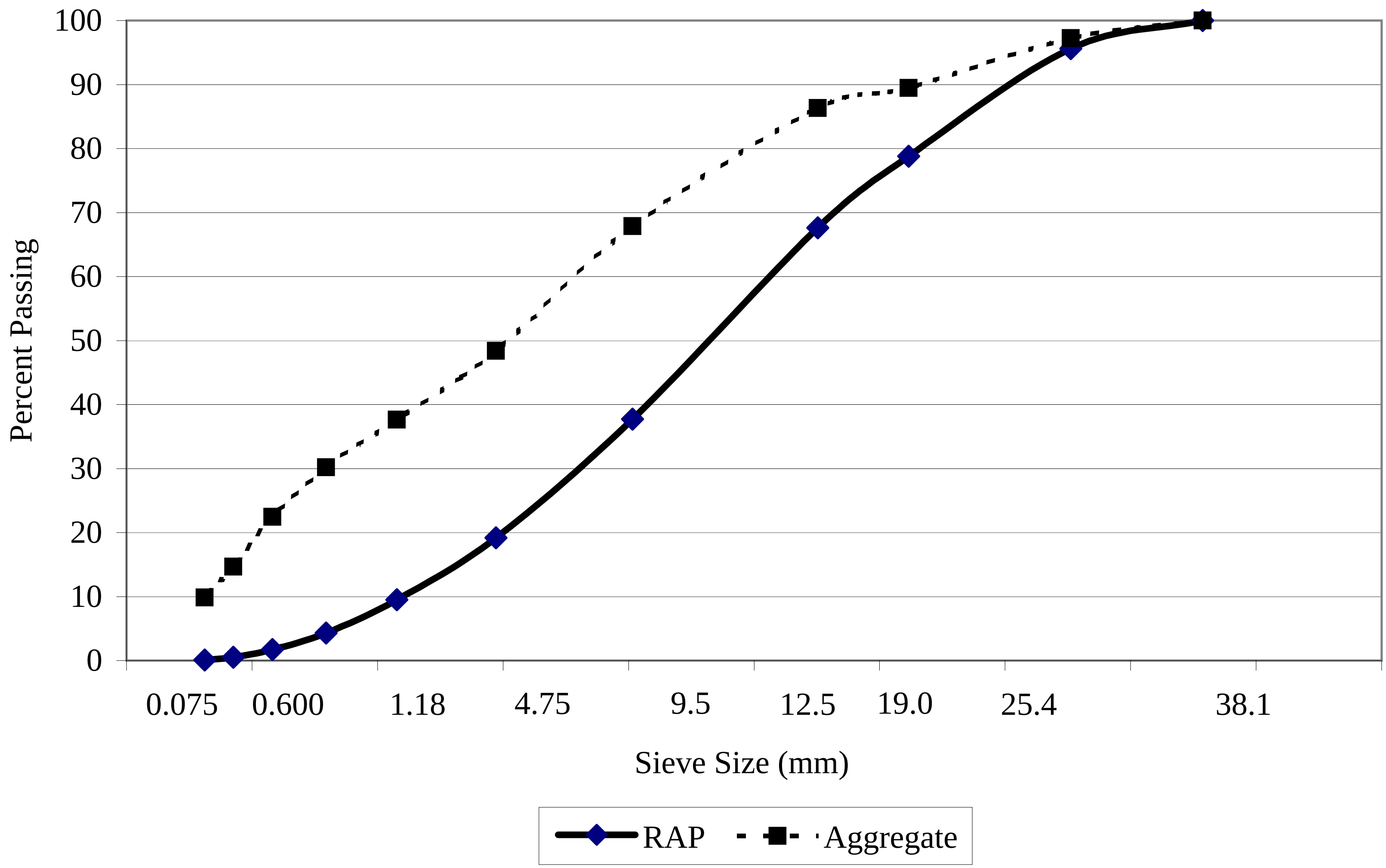

Figure 8. RAP and Extracted Aggregate Gradation, Site 3

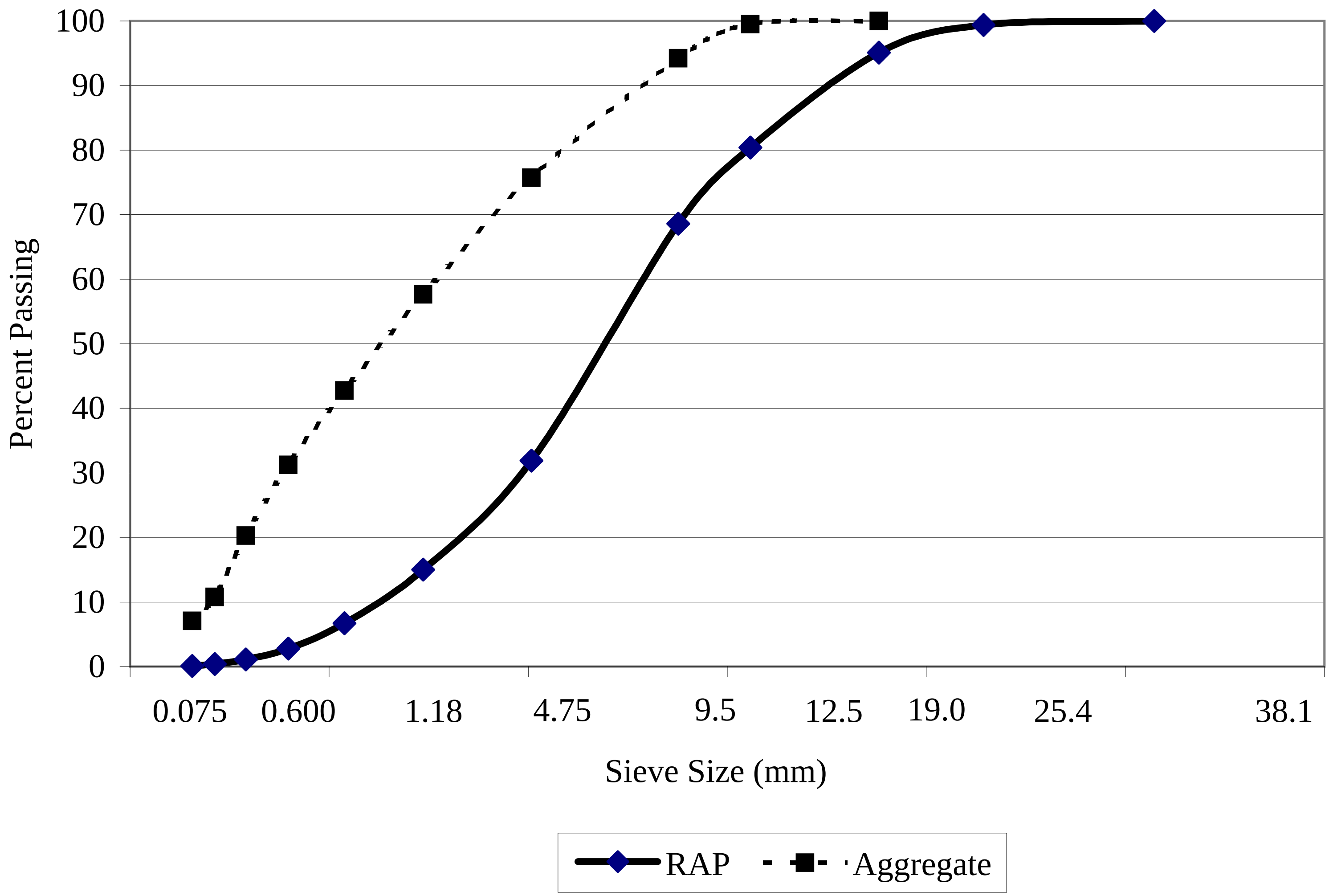

Figure 9. RAP and Extracted Aggregate Gradation, Site 4

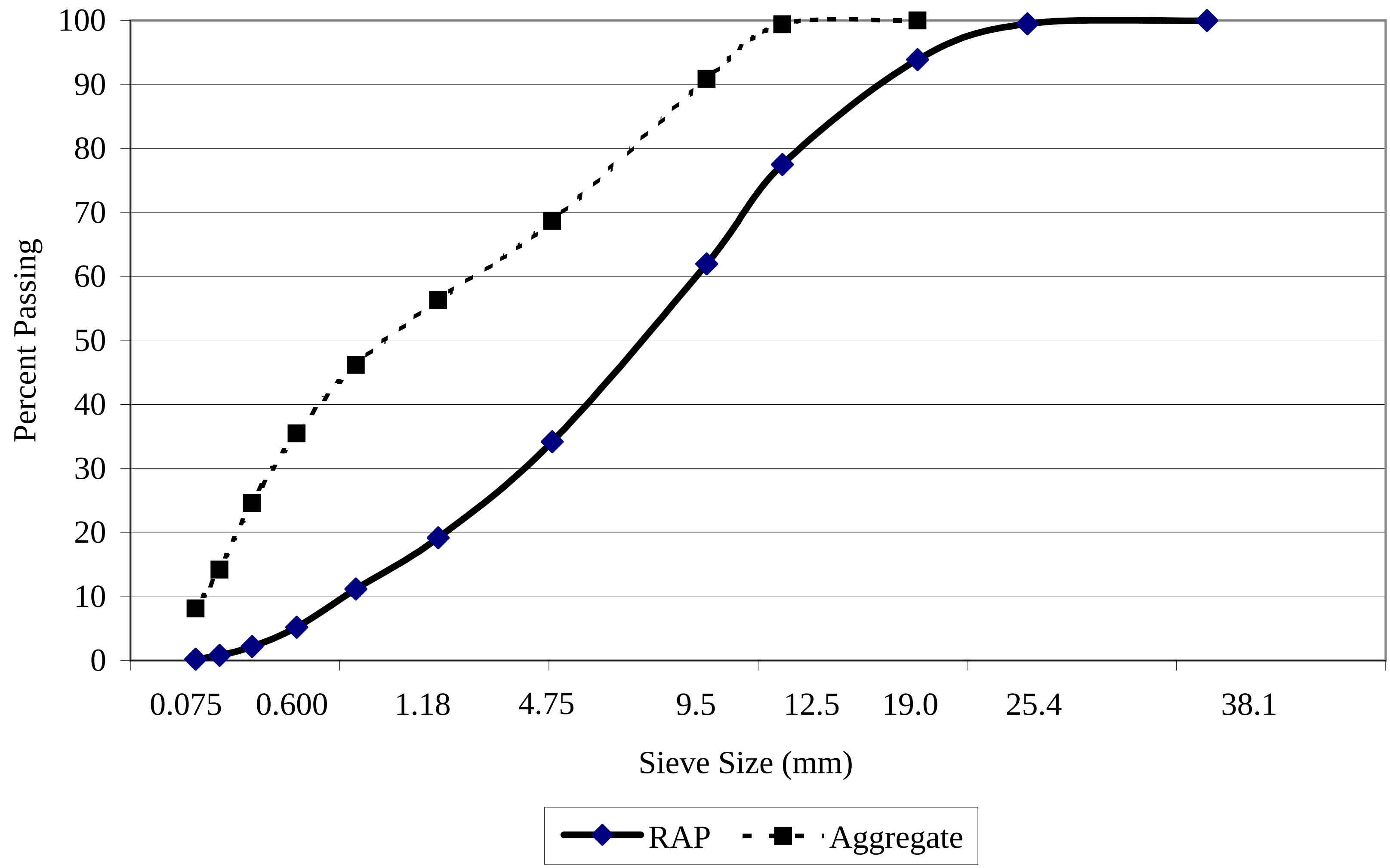

Figure 10. RAP and Extracted Aggregate Gradation, Site 5

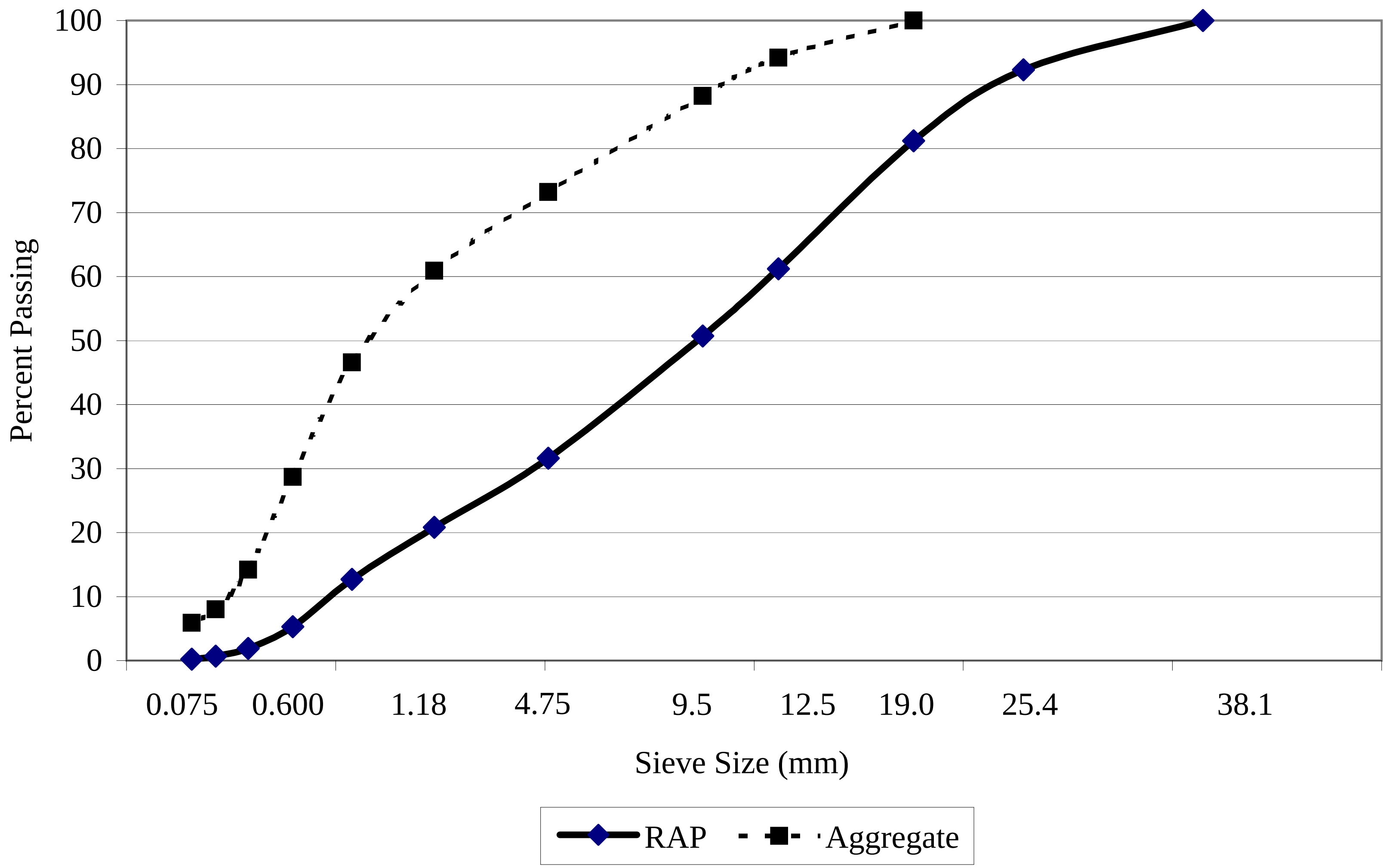

Figure 11. RAP and Extracted Aggregate Gradation, Site 6

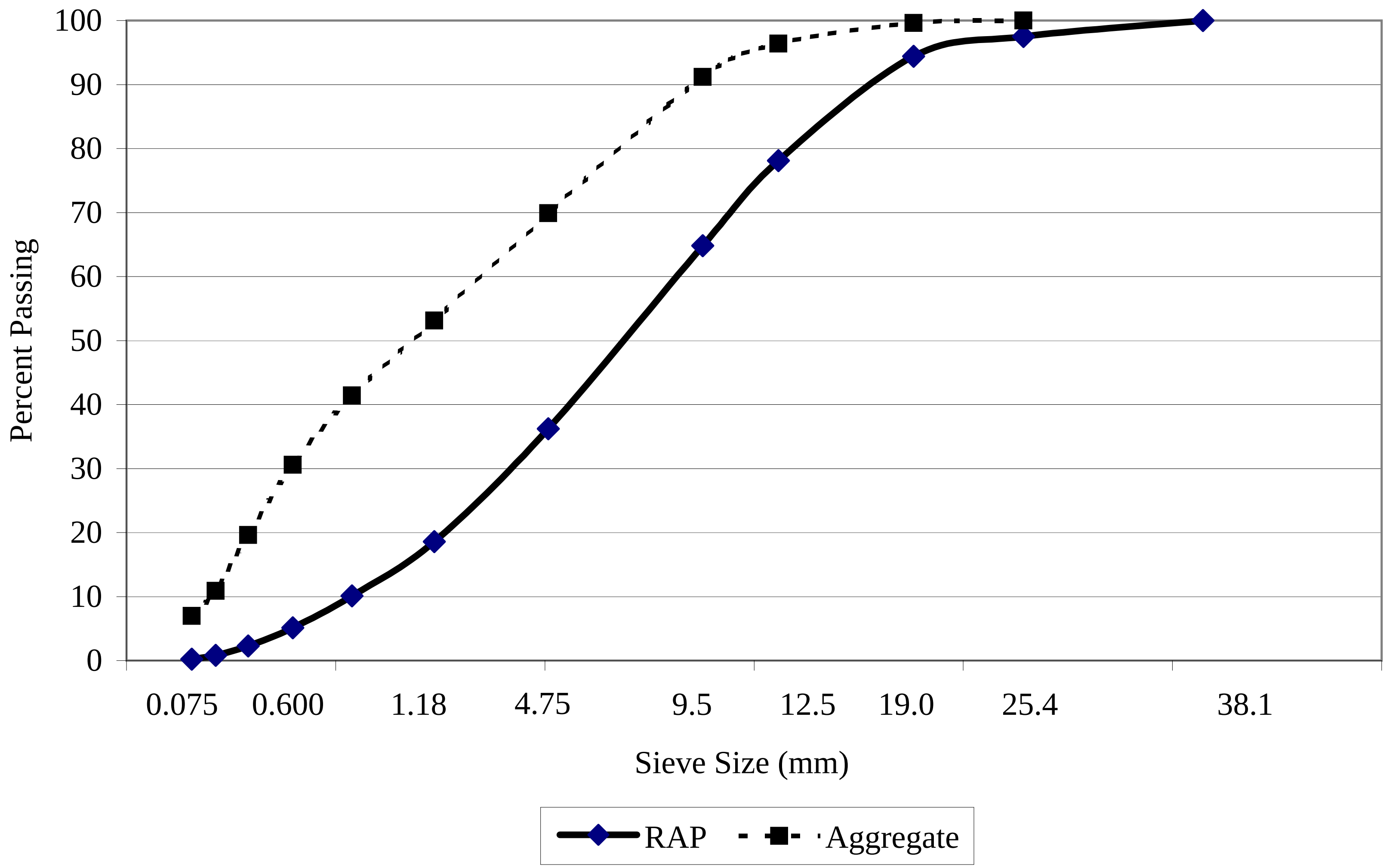

Figure 12. RAP and Extracted Aggregate Gradation, Site 7

Compaction of Samples

All samples were mixed and compacted in general accordance with the recommendations of Lee [(1)] and AASHTO TP 4. Gradation of the RAP, EAC and mixing water contents were based on field test results. Table 7 shows the EAC, mix water and lime slurry content used for each site. After mixing, the samples were allowed to cure for 0, 30, 60 and 120 minutes to ensure that the emulsion had broken. After the initial cure time, the samples were compacted to 50 gyrations at ambient temperatures using an SGC in accordance with AASHTO TP 4. The height of the samples were monitored and recorded continuously during compaction. After compaction, the samples were extruded from the compaction mold and placed in a forced draft oven at 60^{o}C (140^{o}F) for 48 hours for final curing.

Table 7. Compaction Additive Contents*

Site State Route	1 KS US-283	2 KS US-24	3[+] NY US-11	4 SD US-281	5 VT RT-2	6 IA K-42	7 AZ N/A
EAC (%)	1.5	2.15	2.2	1.1	1.5	2.0	2.5
Mix Water (%)	N/A	N/A	2.4	3.0	2.0	1.5	2.0
Lime Slurry							
Total (%)	4.5	4.2	N/A	N/A	N/A	N/A	N/A
Solids (%)	1.5	1.6	N/A	N/A	N/A	N/A	N/A

*Based on dry mass RAP N/A = Not Applicable.

[+] Based on dry mass RAP + add-stone

After the 48-hour oven cure, the samples were removed from the oven and allowed to cool to room temperature. Next, the bulk specific gravity was determined in accordance with AASHTO T 166. Some of the samples were tested for bulk specific gravity using a CoreLok™ device. The results are shown in table 8. Based on the recorded heights and final bulk specific gravity, the bulk specific gravity with each compaction revolution was calculated as specified in AASHTO TP 4. The results are shown in the appendix.

Table 8. Results of Bulk Specific Gravity Testing

Site	State	Route	Sample	Cure Time (mim)	Bulk Specific Gravity AASHTO T 166	CoreLok CoreGravity (TM)
1	KS	US-283	1	0	2.170	*
1	KS	US-283	2	0	2.156	*
1	KS	US-283	1	30	2.131	*
1	KS	US-283	2	30	2.130	*
1	KS	US-283	1	60	2.114	*
1	KS	US-283	2	60	2.125	*
1	KS	US-283	1	120	2.129	*
1	KS	US-283	2	120	2.128	*
2	KS	US-24	1	0	2.128	2.131
2	KS	US-24	2	0	2.127	2.130
2	KS	US-24	1	30	2.114	2.114
2	KS	US-24	2	30	2.123	*
2	KS	US-24	1	60	2.118	2.120
2	KS	US-24	2	60	2.127	*
2	KS	US-24	1	120	2.132	*
2	KS	US-24	2	120	2.122	*
3	NY	US-11	1	0	2.225	2.211
3	NY	US-11	2	0	2.247	2.239
3	NY	US-11	1	30	2.233	2.223
3	NY	US-11	2	30	2.236	*
3	NY	US-11	1	60	2.220	2.212
3	NY	US-11	2	60	2.227	*
3	NY	US-11	1	120	2.216	*
3	NY	US-11	2	120	2.216	*
4	SD	US-281	1	0	2.097	2.086
4	SD	US-281	2	0	2.101	2.077
4	SD	US-281	1	30	2.095	2.086
4	SD	US-281	2	30	2.082	*
4	SD	US-281	1	60	2.085	2.061
4	SD	US-281	2	60	2.088	*
4	SD	US-281	1	120	2.079	*
4	SD	US-281	2	120	2.084	*

* Test not performed.

Table 8 (Con't.). Results of Bulk Specific Gravity Testing

Site	State	Route	Sample	Cure Time (mim)	Bulk Specific Gravity AASHTO T 166	Bulk Specific Gravity CoreLok CoreGravity (TM)
5	VT	RT-2	1	0	2.123	2.120
5	VT	RT-2	2	0	2.121	2.110
5	VT	RT-2	1	30	2.103	2.098
5	VT	RT-2	2	30	2.091	*
5	VT	RT-2	1	60	2.110	*
5	VT	RT-2	2	60	2.119	*
5	VT	RT-2	1	120	2.113	*
5	VT	RT-2	2	120	2.109	*
6	IA	K-42	1	0	2.121	2.096
6	IA	K-42	2	0	2.124	2.090
6	IA	K-42	1	30	2.101	2.086
6	IA	K-42	2	30	2.119	*
6	IA	K-42	1	60	2.116	2.102
6	IA	K-42	2	60	2.130	*
6	IA	K-42	1	120	2.142	*
6	IA	K-42	2	120	2.132	*
7	AZ	N/A	1	0	2.124	2.113
7	AZ	N/A	2	0	2.124	2.108
7	AZ	N/A	1	30	2.096	2.084
7	AZ	N/A	2	30	2.110	*
7	AZ	N/A	1	60	2.118	2.108
7	AZ	N/A	2	60	2.104	*
7	AZ	N/A	1	120	2.118	*
7	AZ	N/A	2	120	2.114	*

* Test not performed N/A = Information not available.

Permanent Deformation

To evaluate resistance to permanent deformation, two samples from each curing condition were tested for resistance to permanent deformation using an APA in the dry mode. The test temperature was approximately one PG grade below the 85 percent reliability maximum pavement temperature determined using LTPPBind version 2.1 software.[14] The results are shown in table 9.

Table 9. Maximum APA Dry Rut Depths

State	Site	Route	Cure (min)	Sample	Temperature (C) LTPP Bind 85th percentile	Temperature (C) Test	Rut Depth (mm)
KS	1	US-283	0	3	54.2	46	4.25
KS	1	US-283	0	7	54.2	46	4.25
KS	1	US-283	30	8	54.2	46	4.95
KS	1	US-283	30	9	54.2	46	5.61
KS	1	US-283	60	1	54.2	46	1.87
KS	1	US-283	60	2	54.2	46	2.15
KS	1	US-283	120	5	54.2	46	3.46
KS	1	US-283	120	6	54.2	46	4.99
KS	2	US-24	0	3	54.1	46	7.40
KS	2	US-24	0	4	54.1	46	8.11
KS	2	US-24	30	2	54.1	46	5.62
KS	2	US-24	30	8	54.1	46	4.80
KS	2	US-24	60	1	54.1	46	5.89
KS	2	US-24	60	6	54.1	46	4.81
KS	2	US-24	120	5	54.1	46	5.16
KS	2	US-24	120	7	54.1	46	5.16
NY	3	US-11	0	3	45.6	42	4.98
NY	3	US-11	0	4	45.6	42	5.45
NY	3	US-11	30	1	45.6	42	10.34
NY	3	US-11	30	5	45.6	42	7.96
NY	3	US-11	60	2	45.6	42	6.48
NY	3	US-11	60	8	45.6	42	7.25
NY	3	US-11	120	6	45.6	42	9.14
NY	3	US-11	120	7	45.6	42	N/T
SD	4	US-281	0	3	49.4	46	7.02
SD	4	US-281	0	4	49.4	46	6.52
SD	4	US-281	30	2	49.4	46	5.28
SD	4	US-281	30	7	49.4	46	5.85
SD	4	US-281	60	1	49.4	46	5.19
SD	4	US-281	60	8	49.4	46	6.44
SD	4	US-281	120	5	49.4	46	6.39
SD	4	US-281	120	6	49.4	46	6.33
SD	4	US-281	120	6	49.4	46	6.13

N/T = Not tested.

Table 9 (Cont.). Maximum APA Dry Rut Depths

State	Site	Route	Cure (min)	Sample	Temperature (C) LTPP Bind 85th percentile	Temperature (C) Test	Rut Depth (mm)
VT	5	Rt-2	0	1	42.3	42	7.25
VT	5	Rt-2	0	4	42.3	42	7.50
VT	5	Rt-2	30	2	42.3	42	8.82
VT	5	Rt-2	30	8	42.3	42	8.55
VT	5	Rt-2	60	5	42.3	42	8.80
VT	5	Rt-2	60	9	42.3	42	7.35
VT	5	Rt-2	120	6	42.3	42	10.26
VT	5	Rt-2	120	7	42.3	42	8.87
IA	6	K-42	0	3	48.6	46	5.04
IA	6	K-42	0	4	48.6	46	4.56
IA	6	K-42	30	6	48.6	46	8.47
IA	6	K-42	30	9	48.6	46	7.69
IA	6	K-42	60	8	48.6	46	6.91
IA	6	K-42	60	10	48.6	46	8.09
IA	6	K-42	120	5	48.6	46	7.41
IA	6	K-42	120	7	48.6	46	N/T
AZ	7	N/A	0	3	59.8	52	5.91
AZ	7	N/A	0	4	59.8	52	6.56
AZ	7	N/A	30	2	59.8	52	5.96
AZ	7	N/A	30	8	59.8	52	4.87
AZ	7	N/A	60	1	59.8	52	5.56
AZ	7	N/A	60	7	59.8	52	5.55
AZ	7	N/A	120	5	59.8	52	7.15
AZ	7	N/A	120	6	59.8	52	7.45

N/A = Information not available.   N/T = Not tested.

Indirect Tensile Strength

The samples that were not tested in the APA were tested for indirect tensile strength in accordance with ASTM D 4123. The results are shown in table 10.

Table 10. Results from Indirect Tensile Strength Testing

State	Site	Route	Cure (min)	Sample	Indirect Tensile Strength (kPa)	(psi)
KS	1	US-281	0	3	390.4	56.62
KS	1	US-281	0	7	407.2	59.06
KS	1	US-281	30	8	374.1	54.25
KS	1	US-281	30	9	360.1	52.22
KS	1	US-281	60	1	304.2	44.11
KS	1	US-281	60	2	333.5	48.37
KS	1	US-281	120	5	293.9	42.62
KS	1	US-281	120	6	339.6	49.26
KS	2	US-24	0	3	238.9	34.65
KS	2	US-24	0	4	181.3	26.29
KS	2	US-24	30	2	225.1	32.65
KS	2	US-24	30	8	259.6	37.65
KS	2	US-24	60	1	232.3	33.69
KS	2	US-24	60	6	271.3	39.35
KS	2	US-24	120	5	306.0	44.37
KS	2	US-24	120	7	277.6	40.27
NY	3	US-11	0	3	211.5	30.67
NY	3	US-11	0	4	230.6	33.44
NY	3	US-11	30	1	216.5	31.40
NY	3	US-11	30	5	188.0	27.27
NY	3	US-11	60	2	232.1	33.66
NY	3	US-11	60	8	188.9	27.40
NY	3	US-11	120	6	213.6	30.98
NY	3	US-11	120	7	209.5	30.38
SD	4	US-281	0	3	290.2	42.08
SD	4	US-281	0	4	316.7	45.93
SD	4	US-281	30	2	352.8	51.17
SD	4	US-281	30	7	350.0	50.76
SD	4	US-281	60	1	320.7	46.52
SD	4	US-281	60	8	384.6	55.78
SD	4	US-281	120	5	369.1	53.53
SD	4	US-281	120	6	372.6	54.03

Table 10 (Cont.). Results from Indirect Tensile Strength Testing

State	Site	Route	Cure (min)	Sample	Indirect Tensile Strength (kN/m^3)	(psi)
VT	5	Rt-2	0	1	266.0	38.58
VT	5	Rt-2	0	4	262.8	38.11
VT	5	Rt-2	30	2	251.5	36.48
VT	5	Rt-2	30	8	289.8	42.03
VT	5	Rt-2	60	5	291.2	42.23
VT	5	Rt-2	60	9	346.7	50.28
VT	5	Rt-2	120	6	330.3	47.90
VT	5	Rt-2	120	7	312.6	45.34
IA	6	K-42	0	3	352.6	51.13
IA	6	K-42	0	4	418.1	60.63
IA	6	K-42	30	6	505.8	73.36
IA	6	K-42	30	9	346.7	50.29
IA	6	K-42	60	8	420.8	61.03
IA	6	K-42	60	10	486.4	70.55
IA	6	K-42	120	5	465.0	67.45
IA	6	K-42	120	7	414.6	60.13
AZ	7	N/A	0	3	291.2	42.23
AZ	7	N/A	0	4	296.7	43.03
AZ	7	N/A	30	2	248.0	35.97
AZ	7	N/A	30	8	272.3	39.49
AZ	7	N/A	60	1	288.1	41.79
AZ	7	N/A	60	7	312.8	45.37
AZ	7	N/A	120	5	311.8	45.22
AZ	7	N/A	120	6	299.4	43.42

N/A = Information not available.

Chapter 5
ANALYSIS OF TEST RESULTS

BULK SPECIFIC GRAVITY

The bulk specific gravity of all gyratory compacted samples was determined in accordance with AASHTO T 166. In addition, some of the samples were tested for bulk specific gravity using the CoreLok™ device in accordance with the manufacturer's recommendations.(12) The results were shown in table 8. AASHTO T 166 recommends those samples that absorb more than two percent moisture be tested in accordance with AASHTO T 275 using paraffin-coated specimens. Paraffin coating renders the sample useless for further testing; therefore, other methods have been proposed, including the use of parafilm, a shrink-wrap plastic, and the CoreLok™ device. Previous research by the author indicated significantly different bulk specific gravity results for HMA mixtures between AASHTO T 166 and parafilm wrapped samples when the air voids were above eight percent, regardless of the percent absorption.(11)

To determine if the CoreLok™ device would yield significantly different bulk specific gravity values from AASHTO T 166, some of the samples were tested for bulk specific gravity using the CoreLok™ device and a paired t-test was performed. The results of the paired t-test indicated no significant difference in the means at a confidence limit of 99 percent ($\alpha = 0.01$). The absorption of the samples using AASHTO T 166 was generally less than two percent, even though the air voids were above eight percent. The SGC generally produces a sample with smooth sides, reducing the absorption, regardless of air void content.

CURE TIME VERSUS COMPACTED DENSITY

There are several mix design methods for cold mixes and all are slightly different.(2) One of the major differences between the methods involves the initial cure time between mixing and compaction. In the field, the mix is usually placed and compacted when only the material on the outside of the windrow has broken, the material on the inside of the windrow has not broken. Most methods require that the mix break before compacting, while others recommend compacting samples immediately. One of the objectives of this study was to evaluate the effect of this initial curing on the mix design compactive effort (N_{design}).

To evaluate the effect of initial cure time on the mix design compactive effort (N_{design}), samples were compacted immediately after mixing and at 30, 60 and 120 minutes after mixing. With the small amount of emulsified asphalt cement used and the black color of the RAP, it was difficult to determine when the mix broke (changed from brown to black). It was hypothesized that after the emulsion broke the viscosity of the mix would increase, thus decreasing the compacted density. The breaking time of the emulsion could then be determined by evaluating the bulk specific gravity versus initial cure time. The plots of initial cure time versus bulk specific gravity for sites 1-7 are shown in figures 13-19. The compacted density was shown in table 8.

It is apparent from figures 13-19 that the initial cure did not have a major effect on bulk specific gravity. An analysis of variance (ANOVA) was performed on the bulk specific gravity results to determine if initial cure time has a significant effect. At a confidence limit of 95 percent ($\alpha = 0.05$), the initial cure time did not have a significant effect on bulk specific gravity. This would indicate that the SGC is a very efficient compactor and the slight change in viscosity of the mix that results from the emulsion breaking did not significantly affect density. Table 11 shows when the samples were judged to have broken based on figures 13-19. Subsequent data analysis was performed on the samples with no initial cure and after breaking (the cure time indicated in table 11). The cure times indicated in table 11 seem reasonable, with the high float and slow set emulsions requiring 60 minutes to break and the recycling agent (site 7) requiring only 30 minutes to break. The CSS-1 used on site 2 broke in 30 minutes. The manufacturer indicated that the emulsion was a special formulation that reduced the breaking time.

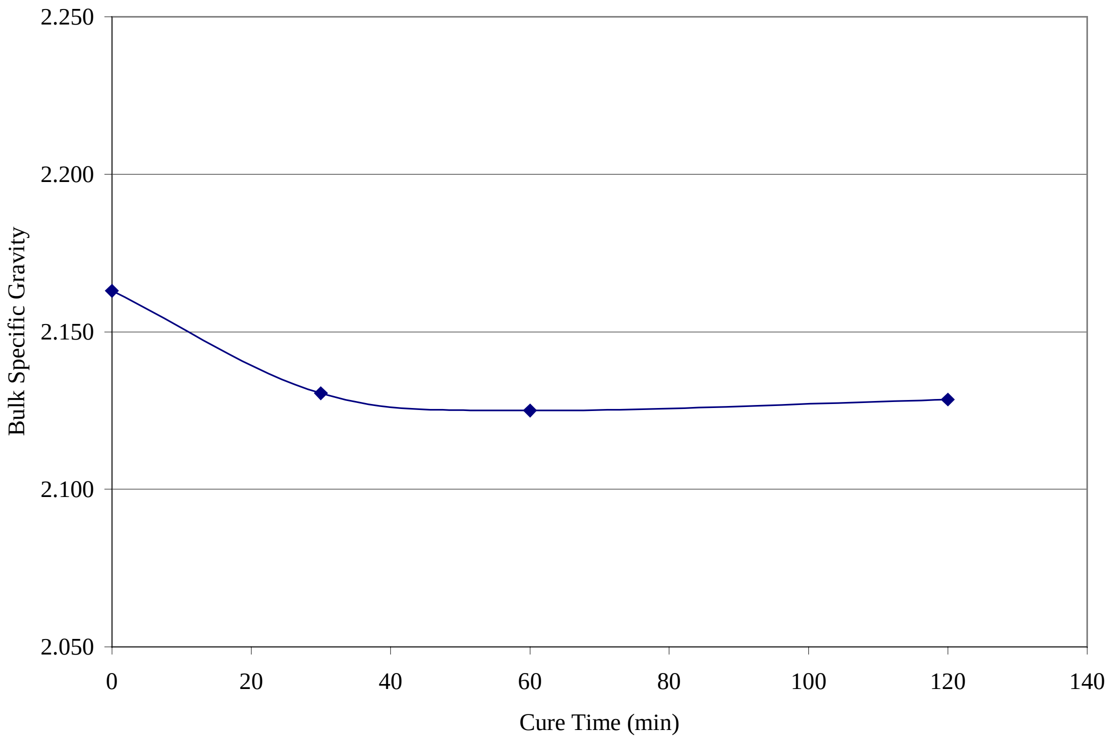

Figure 13. Cure Time versus Bulk Specific Gravity, Site 1

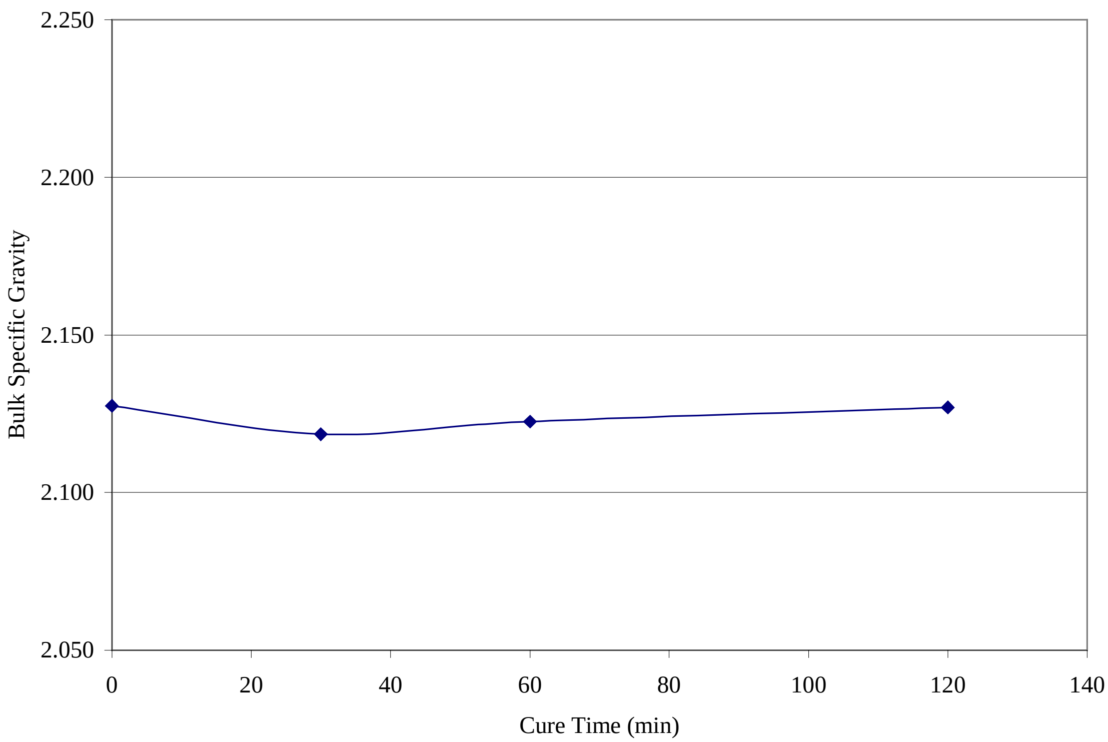

Figure 14. Cure Time versus Bulk Specific Gravity, Site 2

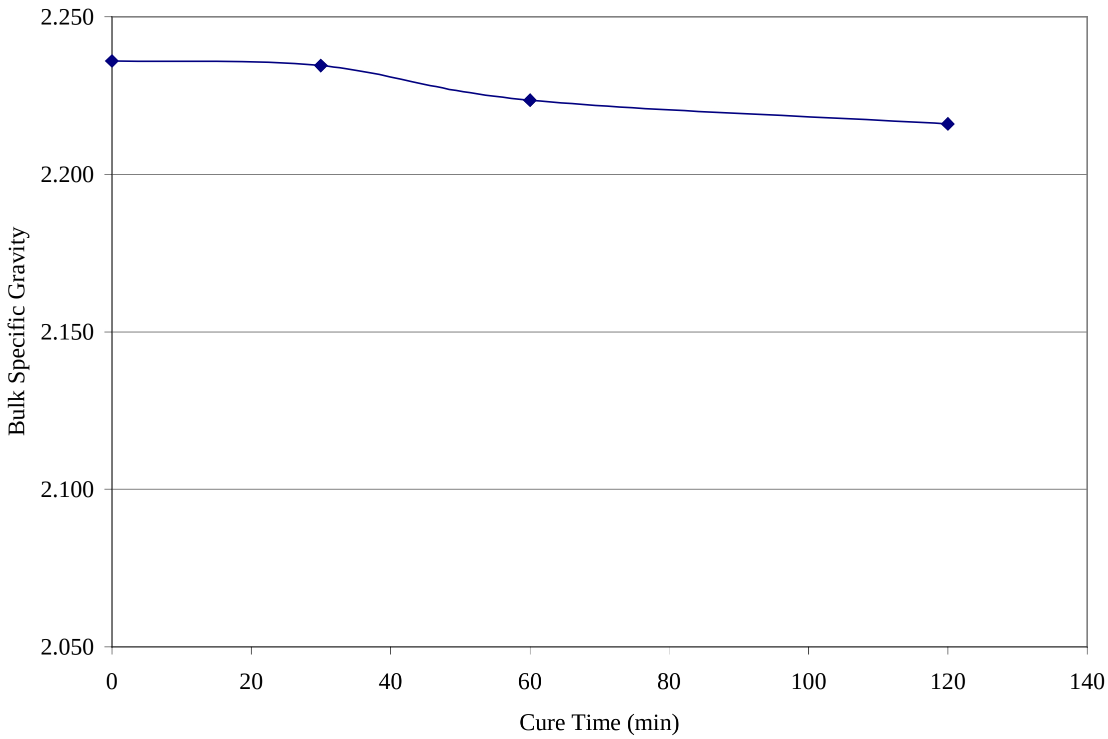

Figure 15. Cure Time versus Bulk Specific Gravity, Site 3

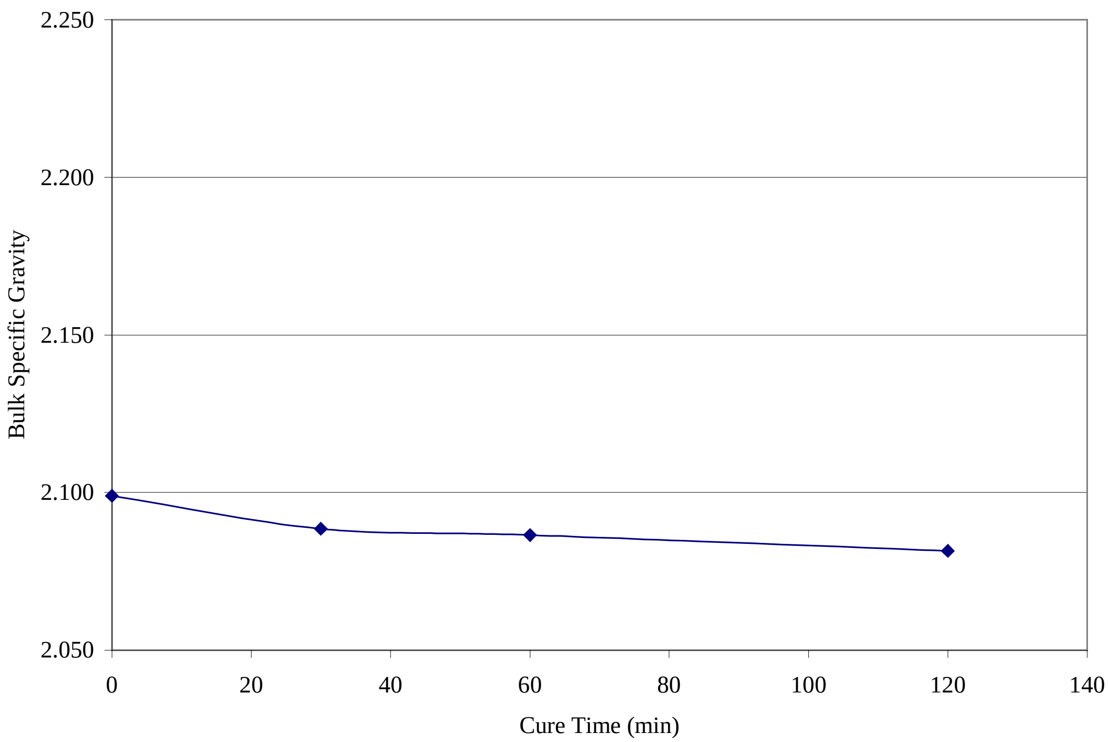

Figure 16. Cure Time versus Bulk Specific Gravity, Site 4

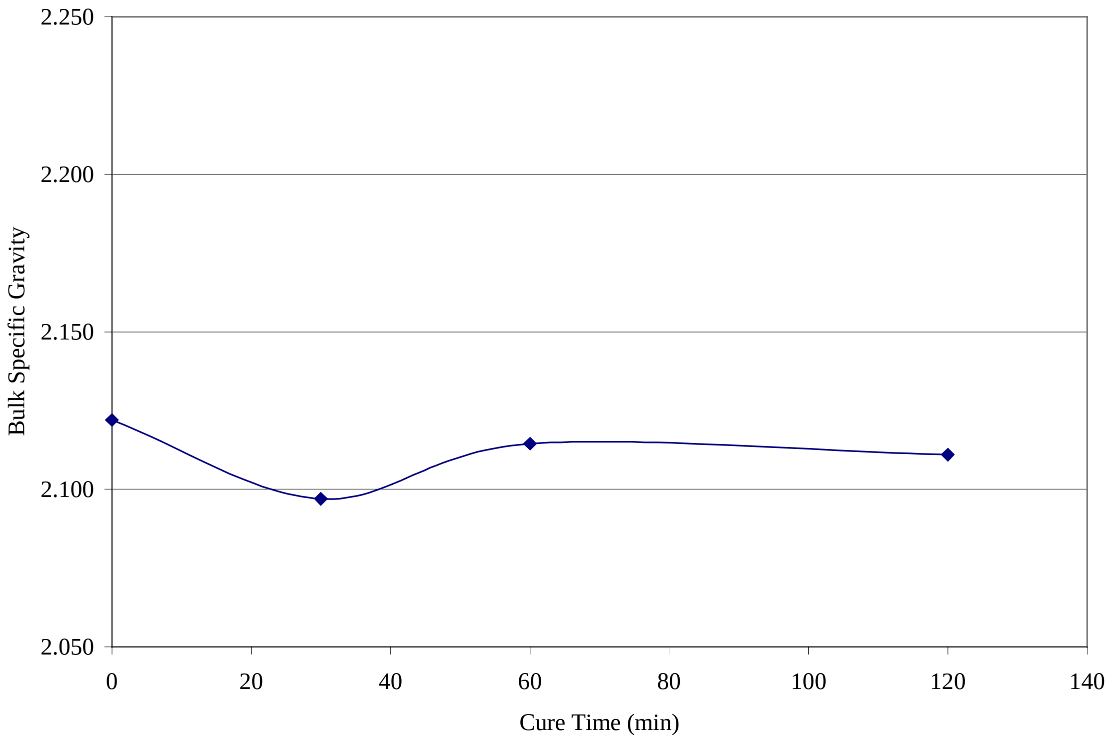

Figure 17. Cure Time versus Bulk Specific Gravity, Site 5

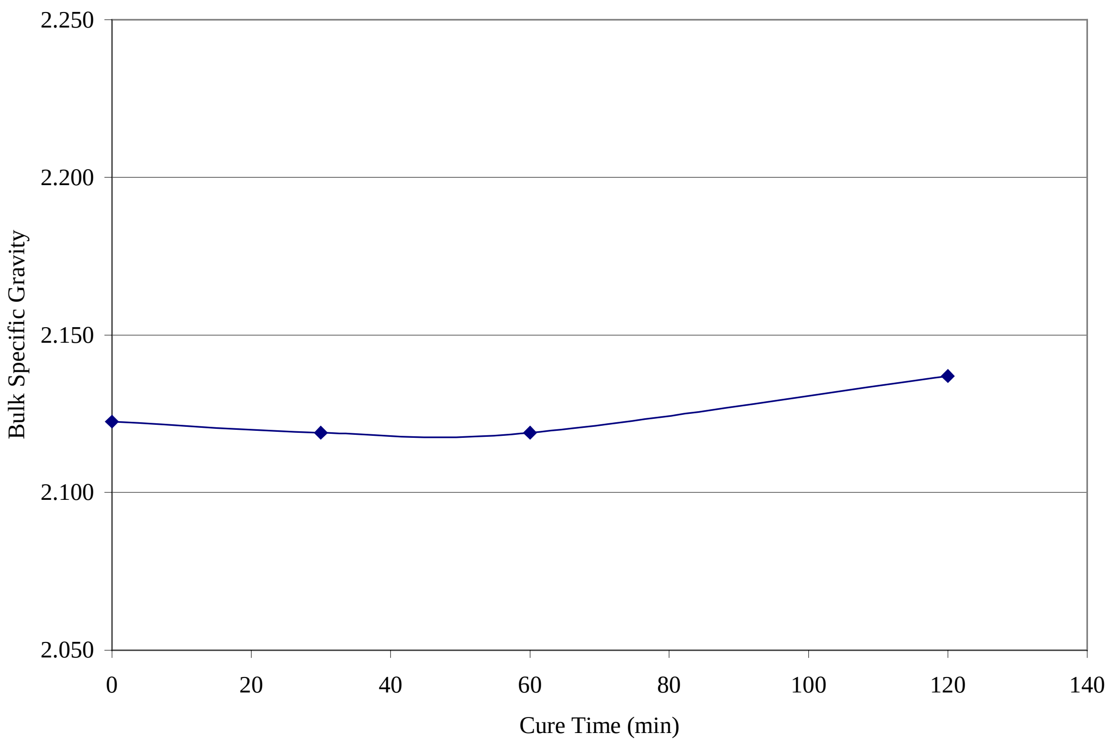

Figure 18. Cure Time versus Bulk Specific Gravity, Site 6

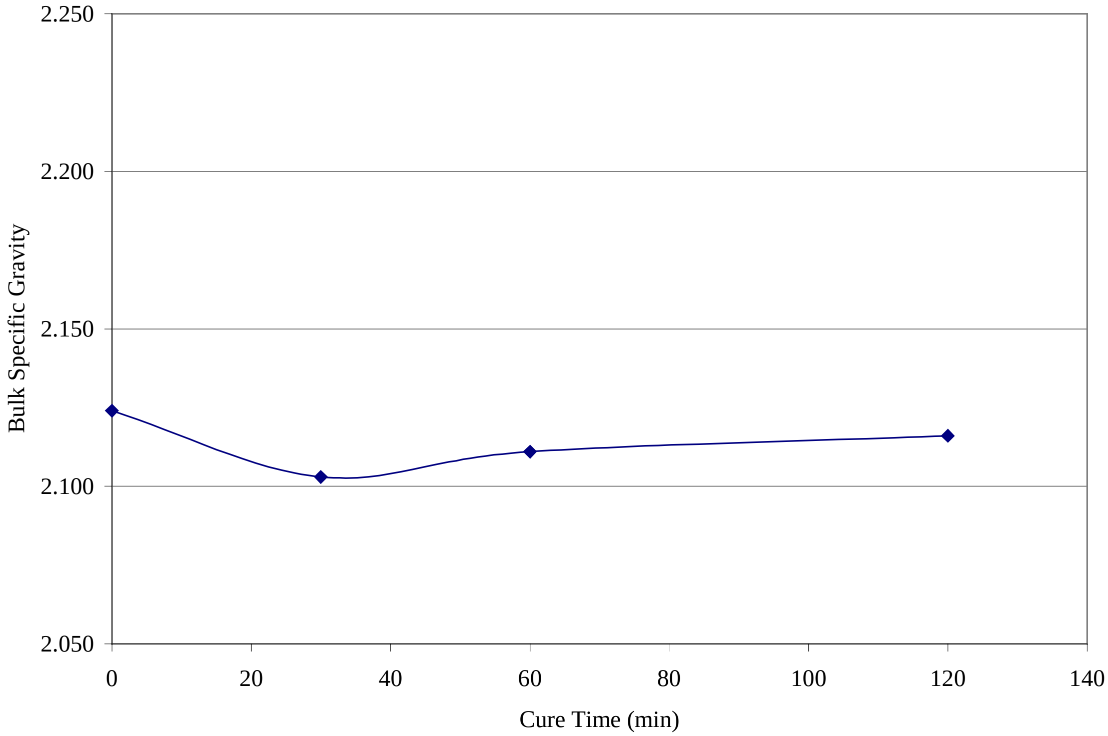

Figure 19. Cure Time versus Bulk Specific Gravity, Site 7

Table 11. Initial Cure Time Required for Breaking

Site	State	Route	Emulsion	Cure Time (min)
1	KS	US-283	CSS-1	60
2	KS	US-24	CSS-1	30
3	NY	US-11	HFMS-2	60
4	SD	US-281	AE200S	60
5	VT	Rt-2	HFMS-2	60
6	IA	K-42	HFE-300	30
7	AZ	N/A	Cyclogene HE	30

N/A = Not available.

GYRATIONS TO FIELD DENSITY

The heights of the samples were monitored during compaction. From the recorded heights and maximum specific gravity of the mix, the percent compaction and density with each gyration were calculated in accordance with AASHTO TP 4. Plots of the percent compaction (% Gmm) with each gyration for the no cure and after breaking samples are shown in figures 20-26.

The majority of the sites had density control with a percent of the laboratory compacted density used as the target value. The required density was some percent of this target value. All sites were well compacted, as shown in table 3. If the field density were used to determine the target value for compaction quality control, then all sites evaluated would have 100 percent compaction. HMA pavements are compacted to 92-94 percent of the mix design density, which corresponds to 96 percent compaction. This calculates to 96 to 98 percent of the laboratory compacted density. Therefore, a target value that results in a field percent compaction of 97 percent was utilized, i.e.:

$$\text{Target value} = \text{field density} / 0.97 \qquad [1]$$

Many existing mix design methods mention that VTM at optimum emulsion content be within a specified range, usually 8-14 percent. However, none of the methods recommend the EAC be adjusted to produce a specific VTM. Therefore, the compactive effort (N_{design}) was evaluated four ways: 1) revolutions to field unit weight, 2) revolutions to a target density, 3) revolutions to 10 percent VTM and 4) revolutions to 12 percent VTM.

To determine the compactive effort (N_{design} gyrations), the gyrations that reproduced the above parameters were determined from plots of gyrations versus density (figures 20-26). The average number of gyrations required and standard deviations for each criterion are shown in table 12. The results indicate that slightly higher compactive effort is required when the samples are allowed to break before compaction. The compactive effort

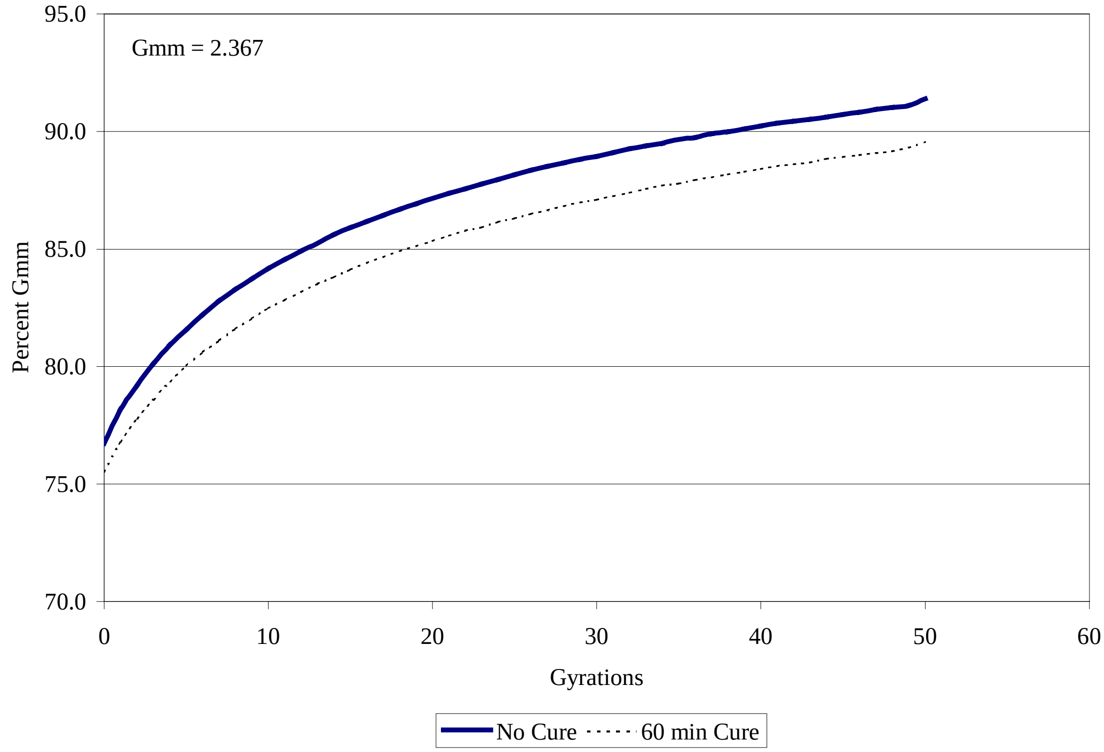

Figure 20. Compactive Effort versus Percent Gmm, Site 1

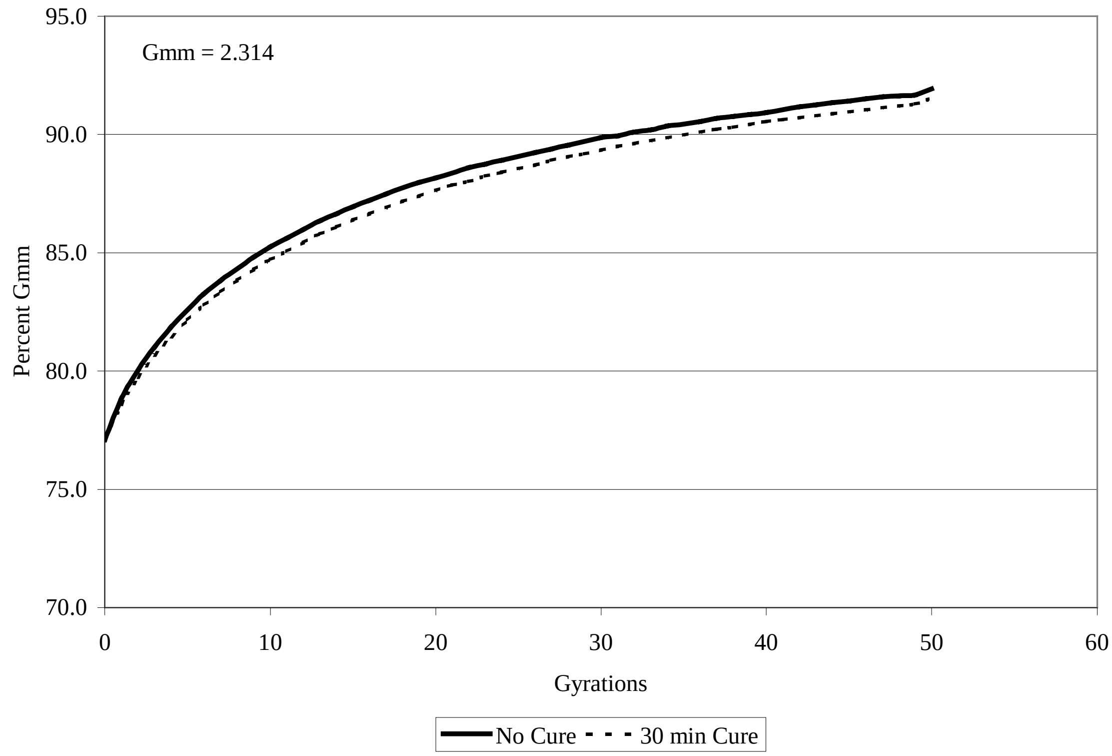

Figure 21. Compactive Effort versus Percent Gmm, Site 2

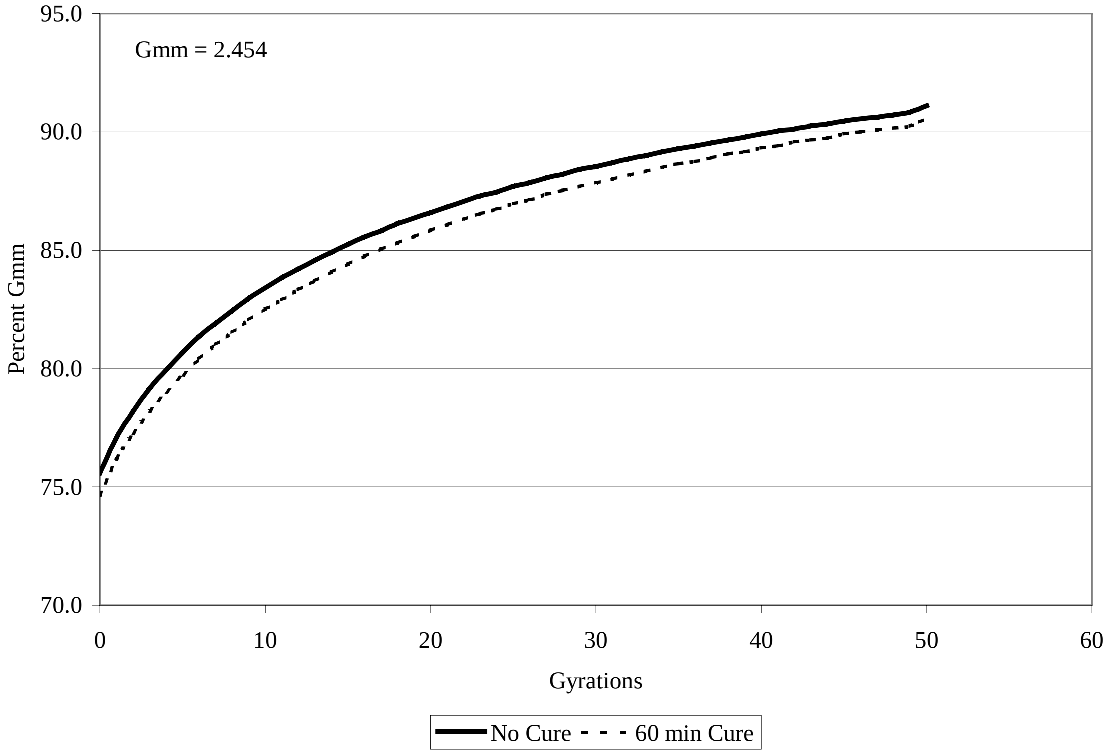

Figure 22. Compactive Effort versus Percent Gmm, Site 3

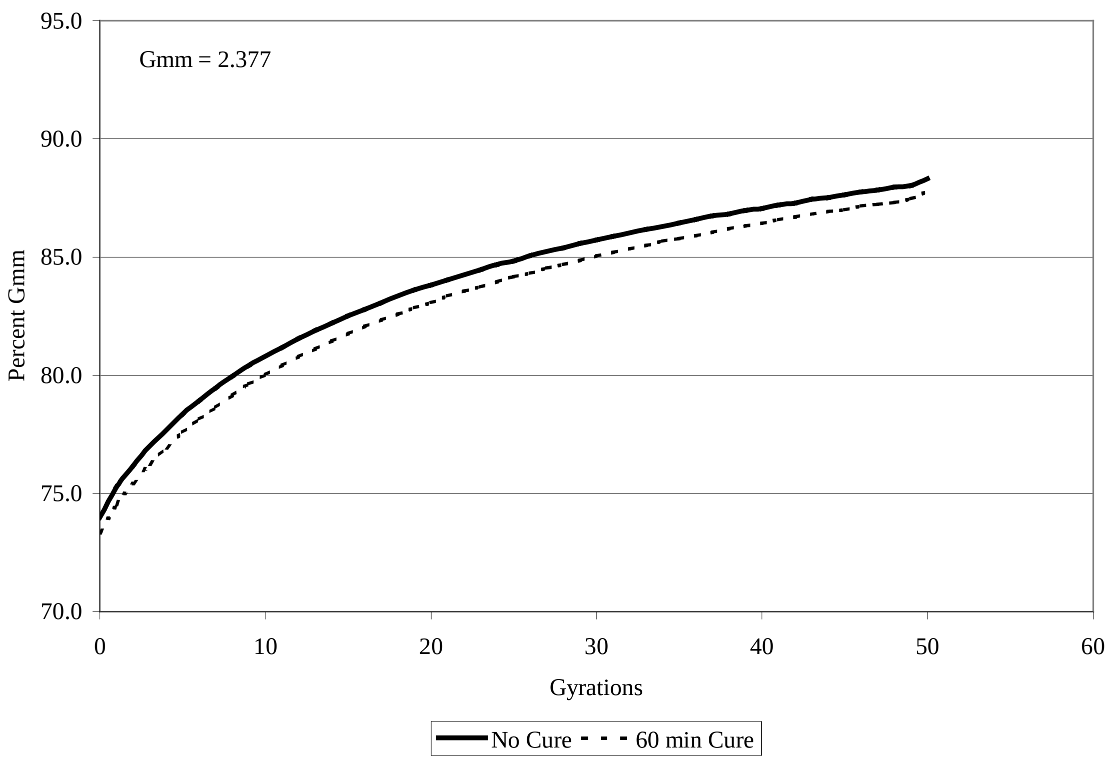

Figure 23. Compactive Effort versus Percent Gmm, Site 4

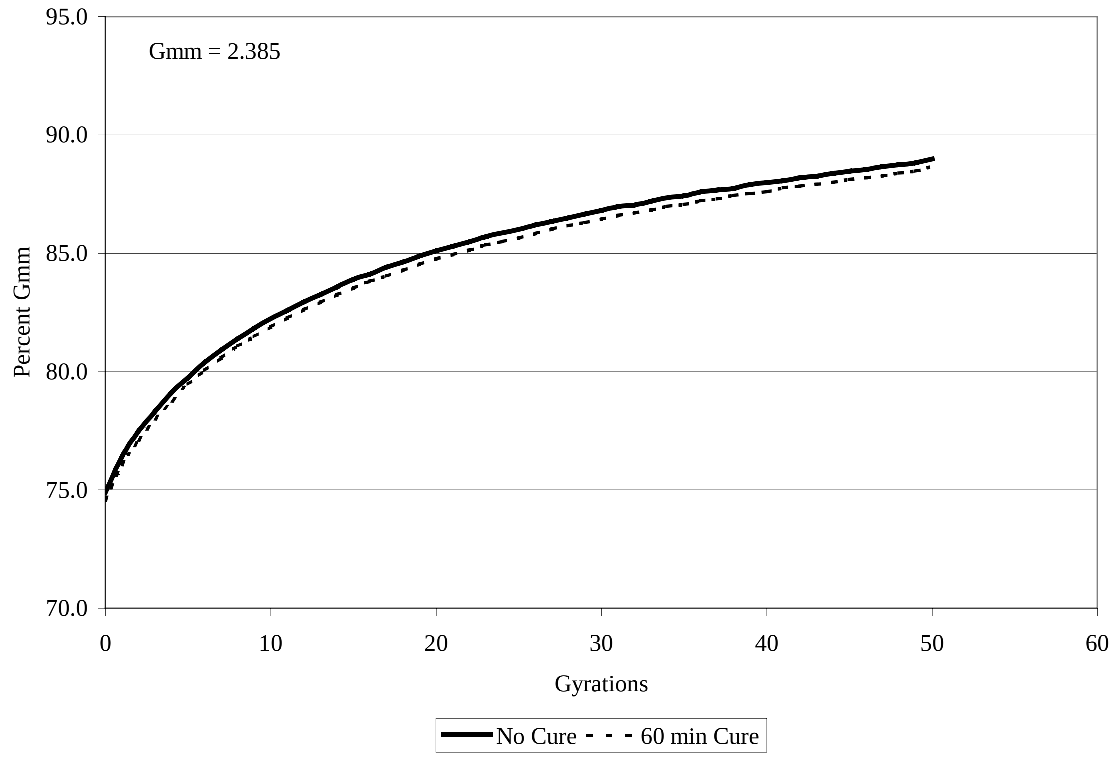

Figure 24. Compactive Effort versus Percent Gmm, Site 5

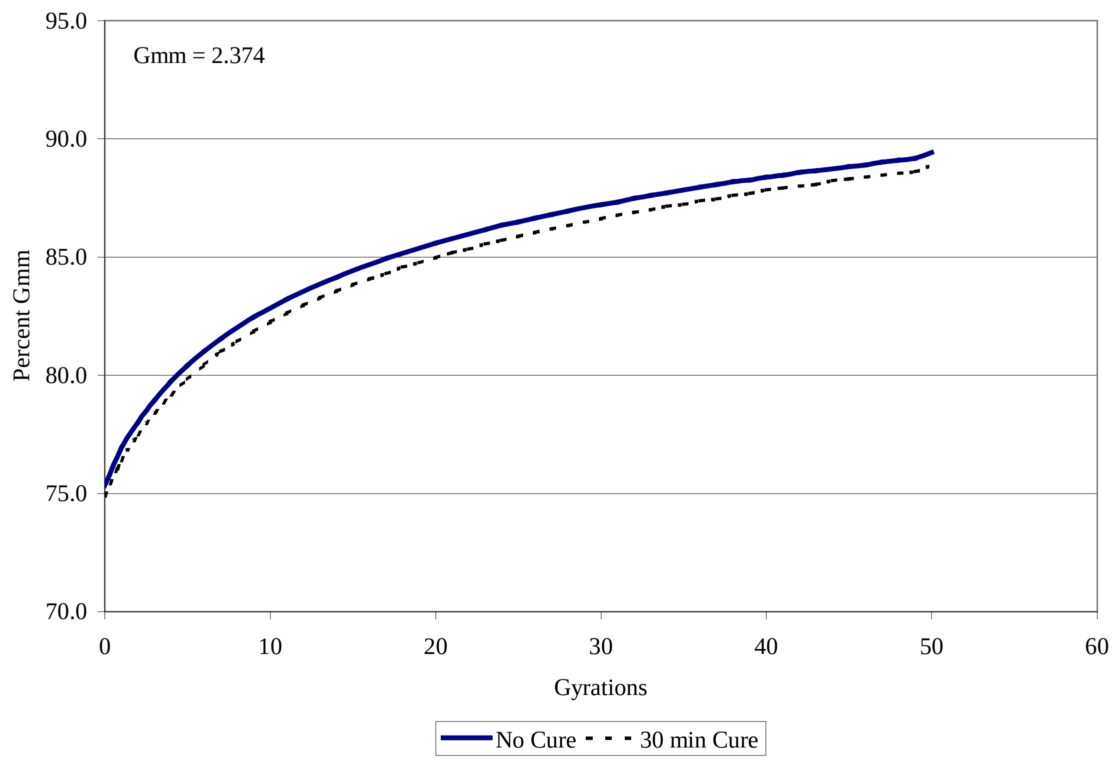

Figure 25. Compactive Effort versus Percent Gmm, Site 6

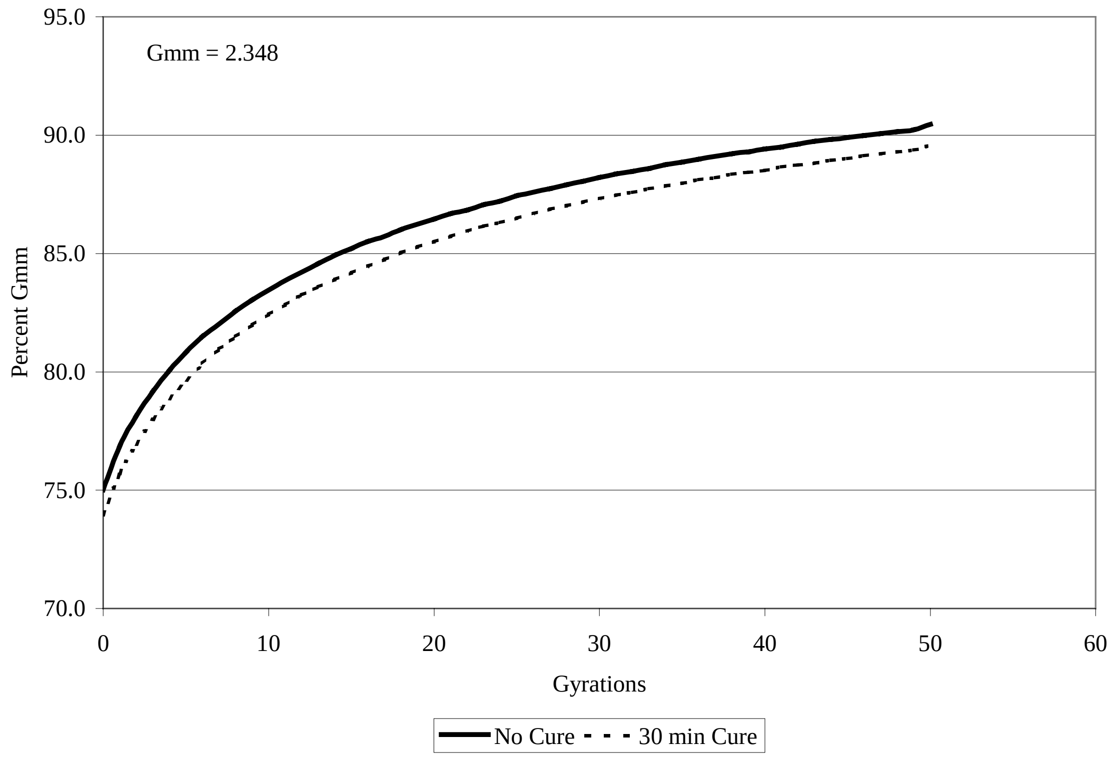

Figure 26. Compactive Effort versus Percent Gmm, Site 7

Table 12. Required N_{design} Compactive Effort (Gyrations)

	No Cure				Break			
Site	Field Density	Target Density	10% VTM	12% VTM	Field Density	Target Density	10% VMT	12% VTM
1	28	50	38	24	43	50	50	36
2	13	25	31	19	15	28	35	22
3	10	18	41	26	13	21	46	31
4	29	50	50	48	33	50	50	50
5	17	32	50	40	19	35	50	44
6	5	10	50	36	6	12	50	42
7	N/A	N/A	46	28	N/A	N/A	50	35
Average	17.0	30.8	43.7	31.6	21.5	32.7	47.3	37.1
Standard Deviation	9.7	16.5	7.4	10.2	13.8	15.4	5.6	9.2

N/A = Field density not available.

required to reproduce the target density and 12 percent VTM were similar, 31 and 32 gyrations, respectively, for the samples compacted without an initial cure, and 33 and 37 for the samples allowed to break. Therefore, it appears reasonable to use a N_{design} compactive effort of 30 gyration for mix design samples that are compacted immediately after mixing and 35 gyrations for samples compacted after breaking.

FACTORS THAT AFFECT N_{design}

As shown in table 12, there was a large range in the compactive effort required to reproduce the field unit weight, 5 to 29 gyrations for no cure samples and 6 to 43 for samples that were compacted after breaking. Physical properties of the RAP and aggregate were evaluated to determine what properties affected field compaction. The factors evaluated and their correlation with the N_{design} number of gyrations are shown in table 13.

Table 13. Correlations Between Physical Properties and Required Compactive Effort.

	Field Density				Target Density			
	No Cure		Break		No Cure		Break	
Parameter	R	n	R	n	R	n	R	n
RAP								
Surface Area	-0.454	14	-0.415	14	-0.413	14	-0.386	14
Fineness Modulus	-0.095	14	-0.128	14	-0.125	14	-0.150	14
Hudson's A Coef.	0.093	14	0.127	14	0.124	14	0.150	14
3:1 Flaky Particles	-0.947	14	-0.967	14	-0.955	14	-0.967	14
2:1 Flaky Particles	-0.474	14	-0.489	14	-0.478	14	-0.483	14
Aggregate								
Surface Area	-0.018	14	-0.052	14	0.004	14	0.023	14
Fineness Modulus	-0.649	14	-0.647	14	-0.648	14	-0.623	14
Hudson's A Coef.	0.676	14	0.668	14	0.677	14	0.653	14
Crushed Faces	-0.164	14	-0.084	14	-0.131	14	-0.089	14
FAA	-0.127	14	-0.065	14	-0.148	14	-0.181	14
Mix								
Indirect Tensile Str.	0.250	14	-0.102	14	0.243	14	-0.180	14

The only parameter that was highly correlated with N_{design} was the percent 3:1 flaky particles in the RAP. The relationships between the number of gyrations required to reproduce the field density and target value are shown in figure 27. The relationships have an R^2 of 0.98 and 0.95, respectively. The relationships indicate that as the percent flaky particles increase, fewer gyrations are required to reproduce the field density. This indicates that the SGC was able to compact flaky mixes to a higher density than conventional construction equipment using reasonable compactive effort. It is generally recognized that the temperature of the RAP, as well as viscosity of the RAP and cutting head speed and direction affect RAP gradation. The data in tables 4 and 5 show that the two sites with the flakiest RAP also had the lowest air and/or RAP temperatures. The effect of the shape of the RAP on performance needs to be investigated before limits on the amount of flaky RAP can be implemented. Insufficient data is available from this study to recommend maximum percentages of flaky RAP and/or minimum pavement temperatures for CIR construction.

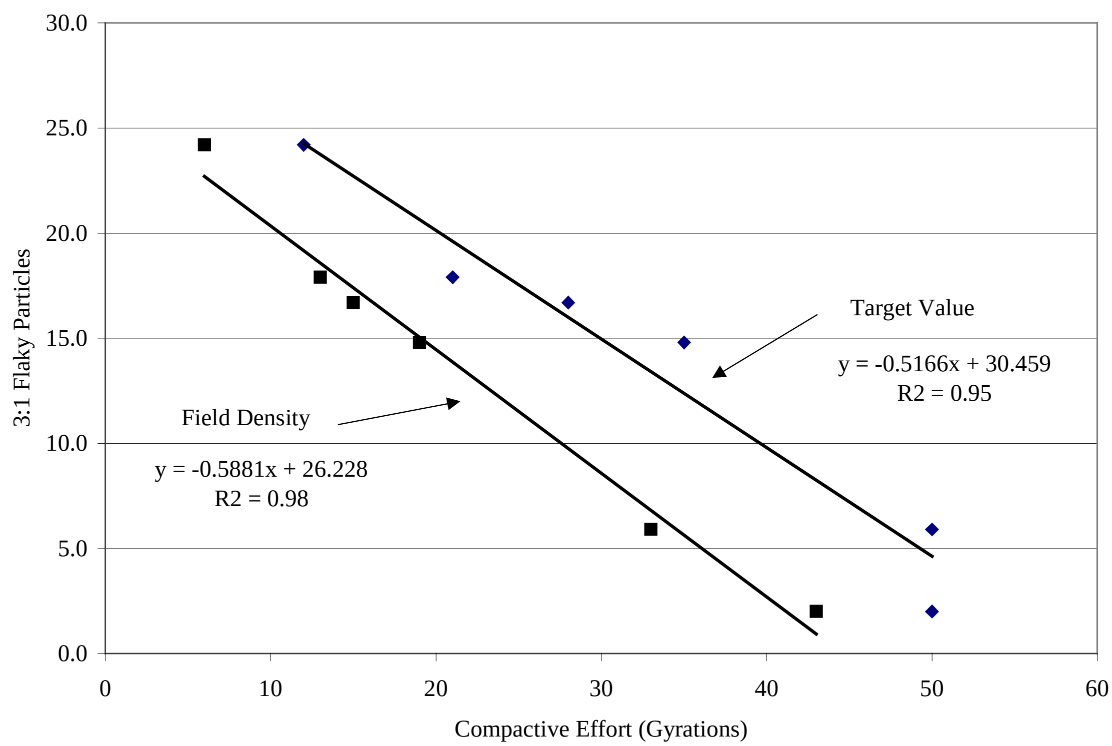

Figure 27. Compactive Effort versus 3:1 Flaky RAP Particles

PERFORMANCE TESTING

The CIR mix samples compacted as a part of this study were tested for indirect tensile strength and resistance to permanent deformation. The samples evaluated were compacted to 50 gyrations, not the field density. Sufficient materials were not available to fabricate additional samples to the field density.

Indirect Tensile Strength

Tensile strength has been related to mixture performance. The results of the indirect tensile strengths were shown in table 10. The correlation between physical properties and indirect tensile strength are shown in table 14. None of the properties evaluated were highly correlated with indirect tensile strength. The initial cure time did not have a significant effect on indirect tensile strength. From table 10, it is interesting to note that the site with the largest percentage of 3:1 flaky coarse aggregate, site 6, also had the highest indirect tensile strength.

Permanent Deformation

The resistance to permanent deformation was determined using the APA in the dry mode. The samples were tested approximately one PG grade below the 85th percentile maximum mix temperature for the layer determined using the LTTPBind version 2.1 software.[(14)] The test temperature was dropped one PG grade to account for the presence of a surface mix over the CIR layer.

Figure 28 shows the average dry APA rut depths for the seven sites. The results indicate higher APA rut depths for the mixes made with high float emulsions. High float emulsions are typically made with softer base asphalts than slow set emulsions. The mixtures with lime as an additive (site 1 and 2) showed some of the lowest rut depths.

Table 14. Correlations Between Physical Properties and Indirect Tensile Strength

	Indirect Tensile Strength					
	No Cure		Break		All	
Parameter	R	n	R	n	R	n
<u>RAP</u>						
Surface Area	-0.367	14	-0.22	14	-0.281	28
Fineness Modulus	0.504	14	0.641	14	0.573	28
Hudson's A Coef.	-0.505	14	-0.645	14	-0.576	28
3:1 Flaky Particles	-0.268	14	0.238	14	0.015	28
2:1 Flaky Particles	-0.250	14	-0.002	14	-0.11	28
<u>Aggregate</u>						
Surface Area	-0.747	14	-0.691	14	-0.707	28
Fineness Modulus	-0.637	14	-0.439	14	-0.519	28
Hudson's A Coef.	0.579	14	0.384	14	0.464	28
Crushed Faces	-0.553	14	-0.716	14	-0.636	28
FAA	-0.120	14	-0.156	14	-0.138	28
<u>Mix</u>						
APA Rut Depth	-0.700	14	0.275	14	-0.052	28

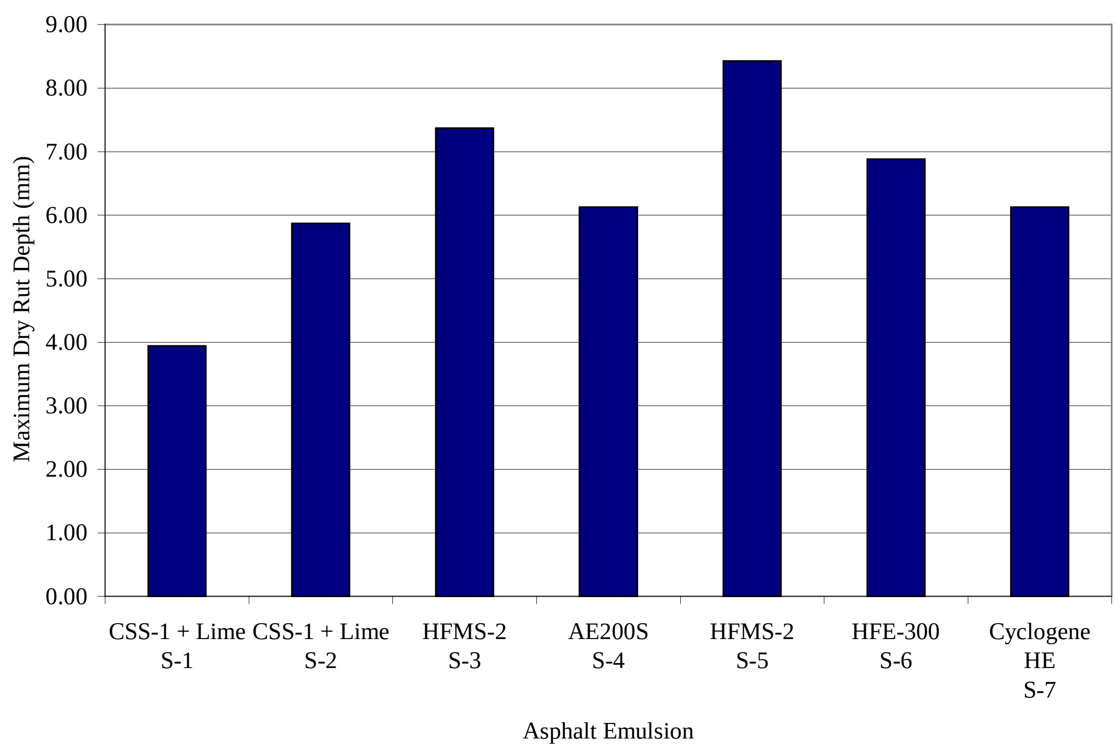

Figure 28. APA Dry Rut Depths

Table 15 shows the correlation between APA rut depths and aggregate and RAP physical properties. The percent 3:1 flaky particles showed a good correlation with rut depth for the samples compacted after break, but not for the samples compacted immediately after mixing. There is no significant difference in rut depth by initial cure. Therefore, the data should be evaluated as a whole and not by cure type, indicating no relationship.

Table 15. Correlations Between Physical Properties and APA Dry Rut Depth

	APA Rut Depth					
	No Cure		Break		All	
Parameter	R	n	R	n	R	n
<u>RAP</u>						
Surface Area	0.417	14	-0.019	14	0.146	28
Fineness Modulus	-0.412	14	0.472	14	0.126	28
Hudson's A Coef.	0.414	14	-0.474	14	-0.127	28
3:1 Flaky Particles	0.080	14	0.766	14	0.488	28
2:1 Flaky Particles	0.211	14	0.032	14	0.099	28
<u>Aggregate</u>						
Surface Area	0.574	14	-0.133	14	0.137	28
Fineness Modulus	0.263	14	0.160	14	0.194	28
Hudson's A Coef.	-0.208	14	-0.167	14	-0.178	28
Crushed Faces	0.185	14	-0.076	14	0.024	28
FAA	-0.346	14	0.264	14	0.027	28
<u>Mix</u>						
Indirect Tensile Str.	-0.700	14	0.275	14	-0.052	28

Chapter 6
CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

Based on the results of this study, the following conclusions are warranted.

1. Mix design samples can be compacted before or after breaking without significantly affecting bulk specific gravity or the performance properties evaluated in this study.
2. Mixtures compacted before the emulsified asphalt cement breaks required 30 gyrations to match the average target value and 12 percent VTM.
3. Mixtures compacted after the emulsified asphalt cement breaks required 35 gyrations to match the average target value and 12 percent VTM.
4. The percent 3:1 flaky particles of the RAP had a significant effect on the N_{design} number of gyrations.
5. The shape of the RAP appears to be partially controlled by the RAP and the air temperature at the time of milling. It is highly likely that viscosity of the RAP and speed and direction of the cutting head affect RAP shape as well. Evaluation of RAP shape and its effect on mix properties and performance were outside the scope of this study.

RECOMMENDATIONS

Based on the results of this study, the following recommendations are made.

1. For CIR mixture design, follow the recommendations of Lee [(1)] and AASHTO TP 4 and use 30 gyrations for N_{design} for samples compacted without the initial cure, and 35 gyrations for samples compacted after the emulsified asphalt cement breaks.
2. The percent 3:1 flaky coarse aggregate particles were shown to affect percent field density and N_{design}. Density of CIR mixtures has been shown to affect some mix parameters that are related to performance. Additional research is

recommended to determine the factors that affect coarse RAP shape and the effect of coarse RAP shape on CIR mixture performance. If RAP shape is shown to have a significant detrimental effect on mixture performance, limits on the percent 3:1 flaky coarse RAP particles should be considered.

APPENDIX – COMPACTION DATA

Table A-1. Bulk Specific Gravity versus Gyrations, Site 1

	Cure Time (min)			
Gyrations	0	30	60	120
0	1.816	1.782	1.787	1.783
1	1.850	1.813	1.818	1.815
2	1.874	1.836	1.841	1.837
3	1.896	1.857	1.860	1.858
4	1.915	1.875	1.878	1.876
5	1.931	1.891	1.894	1.893
6	1.946	1.906	1.908	1.907
7	1.959	1.918	1.920	1.919
8	1.971	1.931	1.931	1.931
9	1.982	1.942	1.942	1.942
10	1.992	1.952	1.952	1.952
11	2.001	1.962	1.960	1.961
12	2.010	1.970	1.968	1.970
13	2.017	1.978	1.977	1.977
14	2.026	1.986	1.983	1.986
15	2.033	1.993	1.991	1.992
16	2.039	2.001	1.998	1.999
17	2.046	2.007	2.003	2.006
18	2.052	2.014	2.009	2.012
19	2.057	2.019	2.015	2.018
20	2.063	2.025	2.020	2.023
21	2.068	2.030	2.025	2.029
22	2.073	2.035	2.030	2.034
23	2.077	2.041	2.034	2.038
24	2.082	2.045	2.039	2.043
25	2.086	2.050	2.042	2.047
26	2.091	2.054	2.047	2.052
27	2.095	2.059	2.050	2.056
28	2.098	2.062	2.055	2.060
29	2.102	2.067	2.059	2.064
30	2.105	2.070	2.061	2.068
31	2.109	2.074	2.065	2.071
32	2.113	2.078	2.068	2.075

Table A-1 (Con't.). Bulk Specific Gravity versus Gyrations, Site 1

	Cure Time (min)			
Gyrations	0	30	60	120
33	2.115	2.081	2.072	2.079
34	2.118	2.084	2.076	2.081
35	2.122	2.087	2.078	2.084
36	2.124	2.091	2.081	2.088
37	2.128	2.093	2.084	2.092
38	2.130	2.096	2.087	2.094
39	2.133	2.099	2.089	2.097
40	2.136	2.102	2.092	2.099
41	2.138	2.105	2.095	2.103
42	2.140	2.108	2.097	2.105
43	2.142	2.109	2.099	2.107
44	2.144	2.113	2.103	2.110
45	2.147	2.115	2.104	2.112
46	2.149	2.117	2.106	2.114
47	2.152	2.120	2.108	2.117
48	2.154	2.123	2.110	2.120
49	2.156	2.125	2.114	2.122
50	2.163	2.130	2.120	2.129

Table A-2. Bulk Specific Gravity versus Gyrations, Site 2

	Cure Time (min)			
Gyrations	0	30	60	120
0	1.785	1.783	1.788	1.780
1	1.825	1.819	1.823	1.817
2	1.852	1.846	1.849	1.843
3	1.875	1.867	1.872	1.866
4	1.894	1.885	1.890	1.886
5	1.912	1.901	1.906	1.903
6	1.927	1.916	1.921	1.918
7	1.940	1.929	1.933	1.931
8	1.952	1.940	1.944	1.942
9	1.963	1.951	1.956	1.953
10	1.973	1.961	1.965	1.963
11	1.982	1.969	1.973	1.973
12	1.990	1.977	1.981	1.981
13	1.999	1.986	1.989	1.989
14	2.005	1.992	1.996	1.996
15	2.012	1.999	2.003	2.003
16	2.019	2.005	2.009	2.009
17	2.025	2.011	2.015	2.016
18	2.031	2.018	2.021	2.022
19	2.036	2.023	2.026	2.027
20	2.041	2.028	2.032	2.032
21	2.045	2.033	2.036	2.037
22	2.051	2.037	2.040	2.042
23	2.054	2.042	2.045	2.047
24	2.058	2.046	2.049	2.051
25	2.061	2.050	2.053	2.055
26	2.065	2.053	2.057	2.059
27	2.069	2.058	2.060	2.063
28	2.073	2.061	2.063	2.067
29	2.076	2.064	2.067	2.070
30	2.080	2.068	2.070	2.074
31	2.082	2.071	2.074	2.077
32	2.086	2.074	2.076	2.079
33	2.087	2.077	2.080	2.083
34	2.091	2.080	2.082	2.085
35	2.093	2.083	2.085	2.089
36	2.096	2.085	2.088	2.092

Table A-2 (Con't.). Bulk Specific Gravity versus Gyrations, Site 2

	Cure Time (min)			
Gyrations	0	30	60	120
37	2.099	2.088	2.091	2.094
38	2.101	2.090	2.093	2.097
39	2.102	2.093	2.096	2.099
40	2.104	2.096	2.097	2.101
41	2.107	2.098	2.100	2.105
42	2.110	2.099	2.102	2.107
43	2.112	2.101	2.105	2.109
44	2.114	2.103	2.107	2.111
45	2.116	2.105	2.109	2.112
46	2.118	2.107	2.111	2.115
47	2.120	2.109	2.113	2.117
48	2.121	2.111	2.115	2.119
49	2.122	2.113	2.117	2.121
50	2.128	2.119	2.122	2.127

Table A-3. Bulk Specific Gravity versus Gyrations, Site 3

	Cure Time (min)			
Gyrations	0	30	60	120
0	1.855	1.846	1.832	1.817
1	1.892	1.883	1.870	1.855
2	1.919	1.910	1.896	1.882
3	1.943	1.933	1.919	1.905
4	1.962	1.953	1.939	1.926
5	1.980	1.971	1.957	1.944
6	1.997	1.987	1.973	1.960
7	2.010	2.002	1.988	1.975
8	2.023	2.016	2.001	1.988
9	2.036	2.028	2.014	2.002
10	2.047	2.039	2.025	2.012
11	2.058	2.050	2.035	2.024
12	2.067	2.059	2.046	2.034
13	2.076	2.068	2.054	2.043
14	2.084	2.077	2.063	2.052
15	2.092	2.085	2.072	2.061
16	2.100	2.093	2.080	2.068
17	2.106	2.100	2.087	2.076
18	2.114	2.108	2.094	2.083
19	2.120	2.113	2.100	2.090
20	2.125	2.120	2.107	2.096
21	2.131	2.126	2.113	2.101
22	2.137	2.131	2.118	2.109
23	2.143	2.136	2.124	2.115
24	2.147	2.142	2.129	2.118
25	2.152	2.146	2.135	2.124
26	2.156	2.152	2.139	2.130
27	2.161	2.156	2.144	2.134
28	2.165	2.161	2.148	2.140
29	2.170	2.165	2.152	2.143
30	2.173	2.169	2.156	2.147
31	2.177	2.173	2.160	2.151
32	2.181	2.177	2.164	2.155
33	2.184	2.181	2.168	2.159
34	2.188	2.185	2.172	2.163
35	2.191	2.187	2.176	2.167
36	2.194	2.191	2.178	2.171

Table A-3 (Con't.). Bulk Specific Gravity versus Gyrations, Site 3

	Cure Time (min)			
Gyrations	0	30	60	120
37	2.198	2.194	2.182	2.173
38	2.201	2.197	2.186	2.177
39	2.204	2.200	2.188	2.181
40	2.207	2.203	2.192	2.183
41	2.210	2.206	2.194	2.187
42	2.212	2.208	2.199	2.189
43	2.215	2.212	2.201	2.193
44	2.217	2.215	2.203	2.195
45	2.220	2.217	2.207	2.197
46	2.222	2.220	2.209	2.202
47	2.224	2.222	2.211	2.204
48	2.227	2.225	2.213	2.206
49	2.230	2.227	2.215	2.208
50	2.236	2.235	2.224	2.216

Table A-4. Bulk Specific Gravity versus Gyrations, Site 4

	Cure Time (min)			
Gyrations	0	30	60	120
0	1.759	1.732	1.743	1.737
1	1.789	1.771	1.771	1.764
2	1.811	1.792	1.793	1.785
3	1.830	1.812	1.812	1.803
4	1.846	1.829	1.828	1.820
5	1.863	1.844	1.844	1.835
6	1.876	1.858	1.857	1.848
7	1.889	1.870	1.869	1.861
8	1.901	1.883	1.881	1.873
9	1.911	1.894	1.893	1.884
10	1.921	1.904	1.902	1.894
11	1.930	1.913	1.911	1.903
12	1.938	1.921	1.920	1.912
13	1.947	1.930	1.928	1.920
14	1.954	1.938	1.936	1.928
15	1.961	1.945	1.943	1.936
16	1.968	1.953	1.951	1.942
17	1.975	1.959	1.957	1.949
18	1.981	1.965	1.963	1.955
19	1.987	1.972	1.969	1.962
20	1.992	1.977	1.974	1.967
21	1.997	1.984	1.981	1.973
22	2.003	1.989	1.986	1.978
23	2.008	1.993	1.990	1.983
24	2.013	1.998	1.996	1.988
25	2.016	2.003	2.001	1.993
26	2.022	2.007	2.004	1.998
27	2.026	2.012	2.009	2.002
28	2.030	2.016	2.013	2.007
29	2.034	2.020	2.017	2.010
30	2.038	2.024	2.021	2.015
31	2.041	2.028	2.025	2.019
32	2.045	2.032	2.028	2.022
33	2.048	2.036	2.032	2.026
34	2.051	2.039	2.036	2.029
35	2.055	2.043	2.039	2.033
36	2.058	2.044	2.042	2.036

Table A-4 (Con't.). Bulk Specific Gravity versus Gyrations, Site 4

	Cure Time (min)			
Gyrations	0	30	60	120
37	2.062	2.048	2.045	2.040
38	2.064	2.052	2.049	2.043
39	2.067	2.054	2.052	2.046
40	2.069	2.057	2.054	2.049
41	2.073	2.061	2.058	2.052
42	2.075	2.064	2.061	2.055
43	2.078	2.066	2.063	2.058
44	2.080	2.068	2.066	2.061
45	2.083	2.071	2.068	2.063
46	2.086	2.074	2.072	2.066
47	2.088	2.076	2.074	2.069
48	2.091	2.078	2.075	2.071
49	2.092	2.081	2.079	2.073
50	2.099	2.089	2.087	2.082

Table A-5. Bulk Specific Gravity versus Gyrations, Site 5

	Cure Time (min)			
Gyrations	0	30	60	120
0	1.786	1.760	1.778	1.774
1	1.822	1.796	1.814	1.808
2	1.848	1.820	1.839	1.832
3	1.868	1.841	1.860	1.852
4	1.886	1.859	1.878	1.870
5	1.902	1.875	1.895	1.886
6	1.917	1.889	1.909	1.900
7	1.929	1.901	1.921	1.913
8	1.941	1.913	1.933	1.924
9	1.951	1.923	1.943	1.935
10	1.961	1.933	1.953	1.944
11	1.969	1.942	1.962	1.953
12	1.978	1.950	1.970	1.961
13	1.985	1.958	1.978	1.969
14	1.993	1.965	1.985	1.977
15	2.001	1.972	1.992	1.984
16	2.006	1.978	1.999	1.990
17	2.013	1.984	2.004	1.996
18	2.018	1.990	2.010	2.003
19	2.024	1.995	2.016	2.008
20	2.030	2.000	2.021	2.013
21	2.034	2.006	2.026	2.019
22	2.038	2.010	2.030	2.022
23	2.044	2.015	2.035	2.027
24	2.047	2.019	2.039	2.032
25	2.051	2.024	2.042	2.036
26	2.055	2.027	2.047	2.040
27	2.059	2.031	2.051	2.043
28	2.063	2.035	2.055	2.048
29	2.066	2.038	2.058	2.052
30	2.070	2.042	2.061	2.054
31	2.074	2.045	2.065	2.058
32	2.076	2.049	2.068	2.061
33	2.079	2.052	2.070	2.065
34	2.083	2.054	2.074	2.068
35	2.085	2.058	2.076	2.071
36	2.089	2.060	2.080	2.073

Table A-5 (Con't.). Bulk Specific Gravity versus Gyrations, Site 5

	Cure Time (min)			
Gyrations	0	30	60	120
37	2.091	2.063	2.082	2.076
38	2.092	2.065	2.085	2.079
39	2.096	2.069	2.087	2.082
40	2.098	2.071	2.089	2.085
41	2.100	2.073	2.093	2.086
42	2.103	2.076	2.095	2.089
43	2.105	2.078	2.096	2.091
44	2.108	2.080	2.098	2.094
45	2.110	2.082	2.101	2.096
46	2.111	2.085	2.103	2.099
47	2.114	2.088	2.105	2.101
48	2.116	2.089	2.108	2.102
49	2.118	2.091	2.110	2.104
50	2.122	2.097	2.115	2.111

Table A-6. Bulk Specific Gravity versus Gyrations, Site 6

	Cure Time (min)			
Gyrations	0	30	60	120
0	1.789	1.777	1.787	1.796
1	1.826	1.813	1.824	1.832
2	1.852	1.839	1.850	1.859
3	1.874	1.861	1.873	1.881
4	1.893	1.879	1.892	1.900
5	1.909	1.895	1.908	1.917
6	1.923	1.909	1.922	1.932
7	1.936	1.922	1.935	1.945
8	1.947	1.933	1.947	1.956
9	1.957	1.943	1.957	1.968
10	1.966	1.952	1.967	1.977
11	1.975	1.961	1.975	1.986
12	1.983	1.969	1.983	1.994
13	1.991	1.977	1.991	2.001
14	1.997	1.983	1.998	2.009
15	2.004	1.990	2.004	2.016
16	2.010	1.996	2.010	2.021
17	2.016	2.001	2.016	2.028
18	2.021	2.008	2.022	2.033
19	2.027	2.012	2.027	2.038
20	2.032	2.017	2.031	2.044
21	2.036	2.022	2.036	2.049
22	2.041	2.026	2.041	2.053
23	2.045	2.031	2.045	2.058
24	2.049	2.034	2.049	2.061
25	2.053	2.039	2.053	2.066
26	2.057	2.042	2.057	2.069
27	2.060	2.046	2.060	2.074
28	2.064	2.049	2.064	2.077
29	2.067	2.053	2.068	2.080
30	2.070	2.056	2.069	2.084
31	2.073	2.060	2.073	2.086
32	2.077	2.062	2.077	2.090
33	2.079	2.065	2.079	2.093
34	2.082	2.069	2.082	2.096
35	2.085	2.071	2.085	2.098
36	2.088	2.074	2.088	2.101

Table A-6 (Con't.). Bulk Specific Gravity versus Gyrations, Site 6

	Cure Time (min)			
Gyrations	0	30	60	120
37	2.090	2.076	2.090	2.104
38	2.093	2.080	2.093	2.107
39	2.095	2.081	2.095	2.109
40	2.098	2.085	2.098	2.111
41	2.100	2.087	2.100	2.114
42	2.103	2.089	2.102	2.116
43	2.104	2.091	2.104	2.118
44	2.106	2.094	2.106	2.121
45	2.108	2.096	2.108	2.123
46	2.110	2.098	2.111	2.125
47	2.113	2.100	2.113	2.127
48	2.115	2.102	2.115	2.129
49	2.117	2.103	2.116	2.131
50	2.123	2.110	2.123	2.137

Table A-7. Bulk Specific Gravity versus Gyrations, Site 7

	Cure Time (min)			
Gyrations	0	30	60	120
0	1.761	1.736	1.747	1.757
1	1.805	1.777	1.788	1.797
2	1.834	1.807	1.818	1.825
3	1.859	1.831	1.842	1.849
4	1.880	1.852	1.863	1.870
5	1.898	1.870	1.881	1.887
6	1.913	1.887	1.897	1.902
7	1.926	1.901	1.910	1.916
8	1.938	1.913	1.924	1.929
9	1.949	1.925	1.935	1.940
10	1.959	1.935	1.945	1.950
11	1.969	1.945	1.955	1.960
12	1.977	1.954	1.963	1.968
13	1.986	1.962	1.971	1.976
14	1.994	1.970	1.979	1.984
15	2.001	1.976	1.986	1.990
16	2.008	1.983	1.992	1.997
17	2.013	1.990	1.999	2.003
18	2.020	1.996	2.005	2.009
19	2.025	2.002	2.011	2.015
20	2.030	2.007	2.016	2.020
21	2.035	2.013	2.020	2.025
22	2.039	2.018	2.026	2.030
23	2.044	2.023	2.031	2.035
24	2.048	2.026	2.035	2.039
25	2.053	2.031	2.039	2.043
26	2.056	2.035	2.042	2.047
27	2.060	2.040	2.048	2.051
28	2.064	2.043	2.051	2.054
29	2.067	2.047	2.055	2.058
30	2.071	2.050	2.057	2.062
31	2.075	2.054	2.061	2.065
32	2.077	2.056	2.065	2.068
33	2.080	2.060	2.066	2.071
34	2.084	2.063	2.070	2.074
35	2.086	2.065	2.074	2.077
36	2.089	2.069	2.076	2.080

Table A-7 (Con't.). Bulk Specific Gravity versus Gyrations, Site 7

	Cure Time (min)			
Gyrations	0	30	60	120
37	2.092	2.071	2.078	2.082
38	2.095	2.074	2.081	2.085
39	2.097	2.076	2.084	2.088
40	2.099	2.078	2.086	2.090
41	2.101	2.082	2.089	2.093
42	2.104	2.084	2.091	2.095
43	2.107	2.085	2.093	2.097
44	2.109	2.088	2.095	2.099
45	2.111	2.090	2.098	2.102
46	2.113	2.093	2.100	2.104
47	2.114	2.095	2.103	2.107
48	2.116	2.096	2.104	2.108
49	2.118	2.098	2.106	2.110
50	2.124	2.103	2.111	2.116

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