FINAL REPORT

STONE MATRIX ASPHALT (SMA) MIXTURES

Project IB-H1, FY92

Prepared by
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Department of Civil Engineering
University of Illinois at Urbana-Champaign

July 1994

Illinois Transportation Research Center
Illinois Department of Transportation

ILLINOIS TRANSPORTATION RESEARCH CENTER

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1.0 CHARACTERIZATION OF STONE MATRIX ASPHALT (SMA) MIXTURES

1.1 INTRODUCTION

The 1990 European Asphalt Study Tour (EAST) brought to the attention of the US. highway industry the Stone Matrix Asphalt (SMA) mixtures which have been used extensively in Europe first to resist studded tire wear, and in recent years in high traffic areas to resist permanent deformation. The current definition of the SMA mixture is:

> *Stone Matrix Asphalt (SMA) is defined as a gap-graded aggregate-asphalt hot mix that maximizes the asphalt cement content and coarse aggregate fraction. This provides a stable stone-on-stone skeleton that is held together by a rich mixture of asphalt cement, filler, and stabilizing agent[1].*

This definition was developed to indicate the important considerations that require careful attention to ensure the mixture produced truly possesses the characteristics of the traditional SMA mixtures being produced in Europe. Table 1-1 illustrates the extent of SMA usage in Europe.

Table 1-1. Applied quantities, applications, and regulations pertaining to European SMA mixes.[4]

	Belgium	Denmark	Finland	France	Germany	Netherlands	Norway	Sweden
Applied Quantity	Nil	Substantial	Small	Nil	Very Large	1.5 million m^2	Small	Fair
Application		Test Sections since 1975 Widely used since 1982	Test Sections since 1986		Normal production since 1970	Since 1984	Since 1985	Test Sections since 1972 Increased use since 1980
Regulations					Standard specifications since 1984	Standard specifications are expected in 1989		Tentative standard specifications

1.2 CURRENT MIXTURE TECHNOLOGY

Today's pavements are subjected to stresses far in excess of what they have experienced in the past. However, the materials being used are being designed and constructed with technology from the past. A major example of this is the design of asphalt concrete mixes which is accomplished with a procedure developed in the 1940's.[1] New design procedures are being developed, such as the AAMAS procedure and the SHRP mix design procedures[2,3]. The AAMAS procedure utilizes more fundamental testing procedures which attempts to characterize structural characteristics of the mixture which will be useful in a structural design of a pavement. The SHRP will attempt to provide a performance based relationship between the mix tests in the laboratory and the finished pavement. Regardless of improved mix design procedures, the fundamental composition of the preferred asphalt concrete mixture has not changed, and the preferred mixture of use today is the dense graded hot mix asphalt concrete (HMA).

The dense graded mixture can have some disadvantages which are being examined in light of recent studies of European mix technology that are calling for the SMA mixture to play a greater role as a surface mixture for asphalt pavements. This mixture was proposed initially to reduce wear under studded tires but a major benefit of the mixture was that it produced a mixture more resistant to rutting than previous mixtures. This benefit is obtained through the emphasis of the stone on stone contact in the mixture. The percentage of stone in the mixture is increased to accomplish this. A typical dense graded asphalt concrete mixture can have little to no stone to stone contact, depending on the gradation. Figure 1-1 illustrates the difference between the internal structure of these two mixes.

The increased stone on stone contact provides a skeleton to carry the load in the SMA mixture. The open voids in this skeleton of stone are filled with the mastic of fine aggregate and asphalt cement, typically resulting in an asphalt rich mixture with an open structure. The load carrying structure in the dense graded mixture comes from the asphalt cement and fine aggregate mastic which floats the stone particles. This mastic in the dense gradation does not provide a positive structure to carry the wheel stress, and could possess less resistance to rutting than the SMA mixture. Figure 1-2 illustrates the gradation differences inherent in a traditional HMA and the recommended FHWA SMA gradation as of the initiation of this project (1992). It is these gradation differences which mandate significantly different approaches to mix design and construction to produce an SMA mixture.

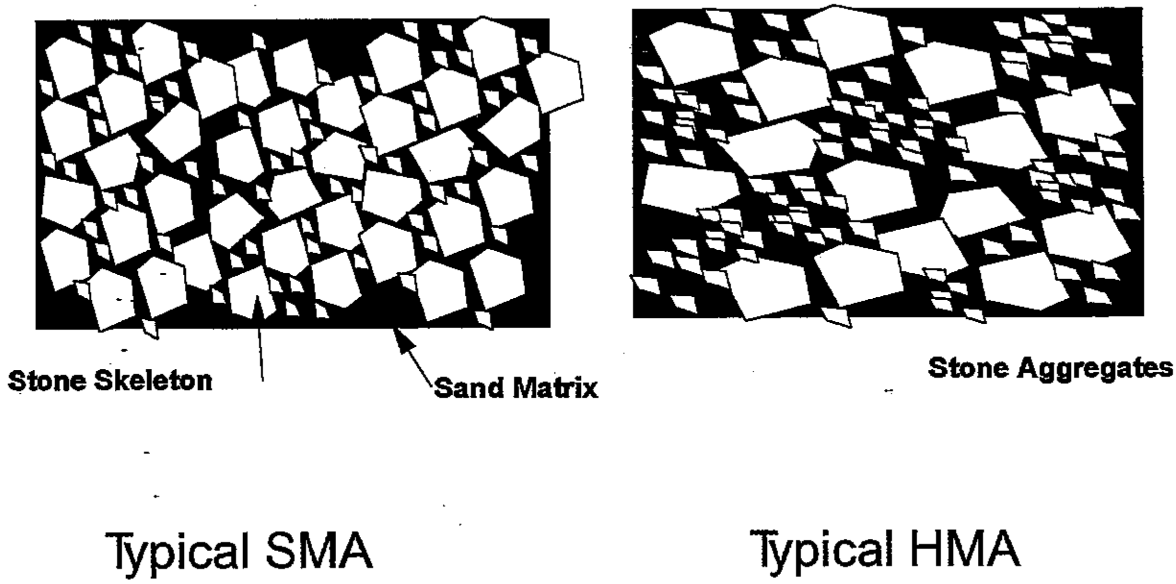

Figure 1-1. Schematic Comparison of internal structure of SMA and HMA mixtures.

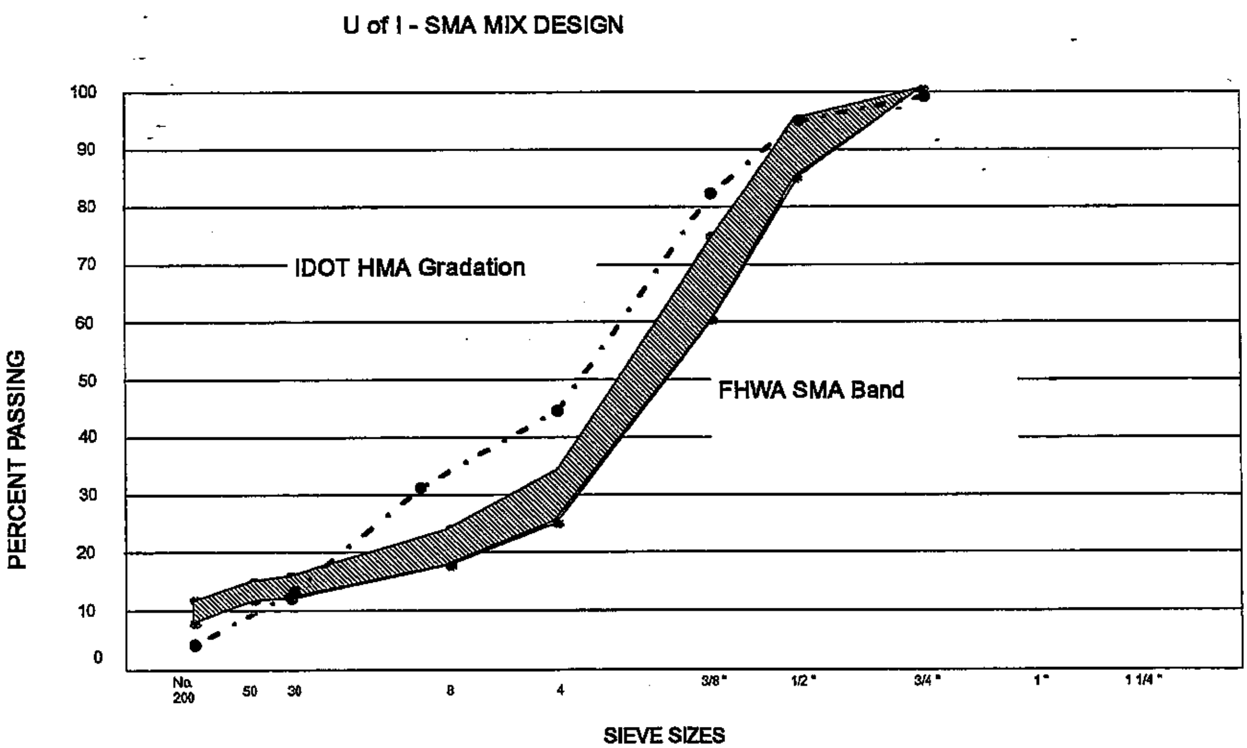

Figure 1-2. Comparison of HMA and FHWA SMA gradation, as of 1992.

1.3 EUROPEAN PRACTICE

1.3.1 Gradations and Aggregates

Figure 1-3 illustrates gradations from Germany and Sweden, two major users of SMA mixtures[4]. There is a significant difference in the aggregate structure that will be provided by these two gradations, although both gradations are used as successful SMA gradations. Evaluation of the structural differences produced in the final mixture must illustrate if the mixes prepared from these two gradations can be considered similar. The difference between an SMA gradation and the traditional HMA gradation was shown in Figure 1-1 and Figure 1-2 which clearly illustrate the production of significantly different materials. The increased

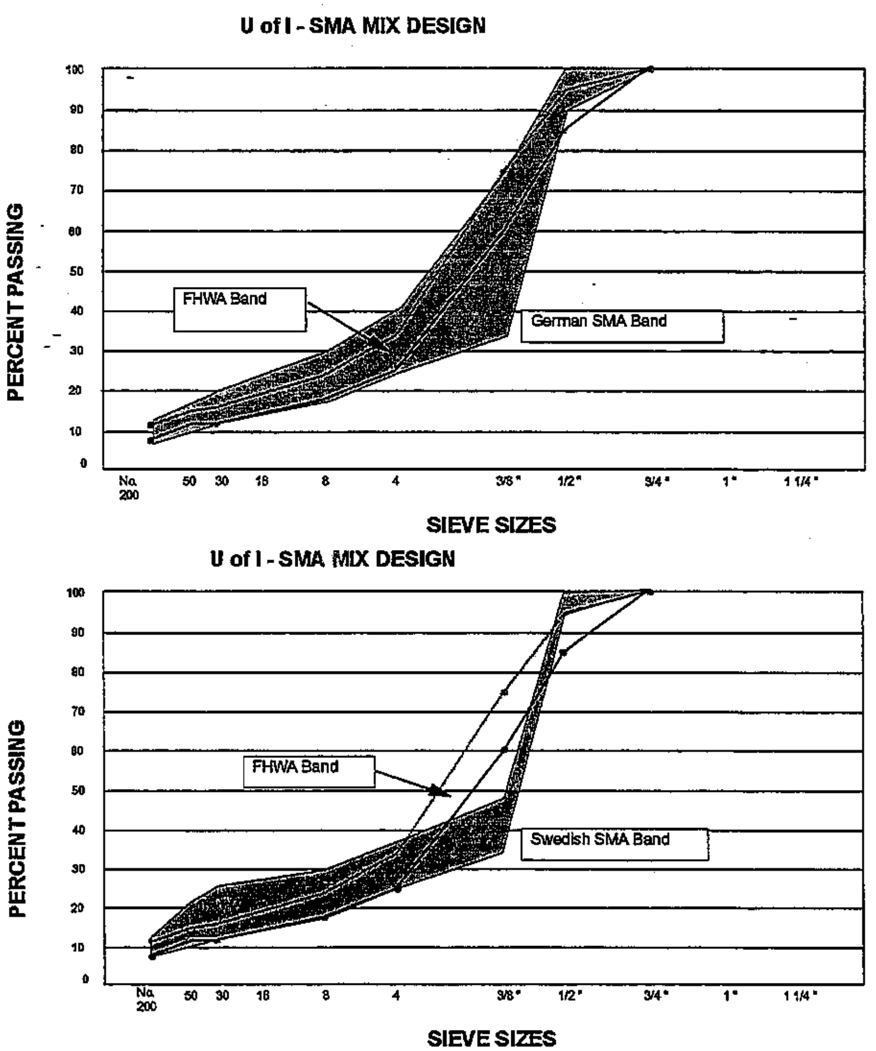

Figure 1-3. Comparison of FHWA recommended and European gradations.

concentration of the larger stone sized aggregates in the SMA is responsible for a mixture with a very high asphalt requirement.

Current recommendation is to use only very hard, sound aggregates in Europe with all crushed material being specified. The particle shape of these aggregate particles should be cubical with no more than approximately one-third being flat or elongated. No particles should have a length to width ratio of more than 5. There is no firm percentage of particle shapes and sizes at present for particles that are not cubical.

The control on the gradations is very good in Europe. Typically a plant will have up to 12 to 15 cold feed bins feeding for one mixture. This practice significantly reduces variability in the final mixture. This high degree control, resulting in less variation on the pavement, may be a major factor in the good performance of these mixtures which should be investigated.

1.3.2 Asphalt Content and Volumetrics

The SMA gradation produces a gap-graded material very similar to open graded mixtures. The presence of a significant amount of fines, however produces a compacted mixture with 3 to 4 percent air voids (VTM). This VTM level is similar to US. practice even though the gradation appears open, a gradation which would typically have some 20 percent voids. Voids in the mineral aggregate (VMA) is not specified nor measured in European practice with the SMA mixture. Compaction of the SMA mixture in the laboratory mix design procedure is performed at 50 blow Marshall compaction which is felt to be satisfactory to achieve lockup of the stone on stone skeleton. The asphalt content required to coat the aggregates and keep volumetrics satisfactory is high, and common practice is to recommend that a minimum of 6 percent by total weight of mix be maintained.

1.3.3 Draindown

This open structure with the increased concentration of larger aggregate sizes in the SMA mixture produces a thicker film of asphalt cement than is normally produced in an HMA. This asphalt binder film must cling to the stone aggregate and not drain off. To accomplish this, the major European experience has been to add a cellulose fiber to the asphalt cement which serves to stiffen the binder This allows for a thick coating of asphalt cement on the aggregate without draindown. Fiber addition is specified at 0.3 percent by weight of mix for cellulose fibers, and 0.4 percent by weight of mix for mineral fibers, as minimums.

The current test methodology for verifying the stabilizing effect of the fiber is to perform a Shellenberger draindown test[1]. This test involves mixing an SMA sample and placing the mixture in a beaker in a heated oven for a specified period of time. The beaker is inverted and the mixture falls out, leaving some asphalt adhering to the glass beaker. The more asphalt binder that is on the beaker, the more draindown has occurred. Excessive draindown will cause problems during transport and laydown as the asphalt binder drains and settles in the bottom of the trucks, resulting in fat and lean spots in the paving layer.

1.4 U. S. PRACTICE

1.4.1 Gradations and Aggregates

The Federal Highway Administration has recommended that European practices be followed where possible. Figure 1-2 illustrates the recommended gradations prepared by FHWA to model European practice[1]. These recommendations more closely approximate the German SMA gradations, and do not have as large an amount of material retained on the 3/8 inch sieve as is common in the Swedish SMA gradation.

Durability and particle shape requirements have typically followed the European recommendations. Los Angeles Abrasion values have been significantly below 40, and all crushed particles have been used in the coarse aggregate. Current recommendations limit this value to 30 or lower. Particle shape limitations require less than 20 percent of the aggregates have a length to width ratio of 3, and less than 5 percent with a ratio of 5. These are in line with European practices.

Different gradations that have been used in SMA mixtures since the EAST study trip are illustrated in Figure 1-4, and Figure 1-5. These gradations indicate a wide variety of gradations that are being lumped together under the SMA heading. There was little or no structural testing of these mixtures to establish the structural differences in the SMA characteristics that may be developed in these disparate mixtures. To place them all under the category of being SMA mixtures implies that performance should be similar. Long term monitoring will be required to definitively show the comparability of these mixtures.

It should be noted that the US. Standard sieve sizes are slightly different than the metric sieves used in European practice. Further, the recommended sizes are not the same.

These differences will produce a different material, even when graded to the same overall gradation curve. The most significant difference arising in the production of slightly different particle sizes which affect the void sizes remaining when they interlock.

The U. S. practice for plant control is to have individual cold feed bins for each aggregate being blended in the final mixture. This does not provide the same level of control over variability in the mixture from loading each bin from different parts of the stockpile. Whether this variability is crucial to the attainment of an SMA mixture has not been examined.

1.4.2 Draindown

Different fibers are being evaluated, and the use of polymer additives is common in U. S. practice with SMA mixtures. The European draindown procedures may not be applicable to polymer modified asphalt binders due to their higher level of adhesion which may produce inconsistent results when placed in a beaker. The use of different fibers or additives should be compared to the results obtained with the European additives. The European additives could result in a significantly different rheological binder at the same degree of draindown. This could result in significantly different binders which could produce different performance in the final mixtures.

1.4.3 Mixture Volumetrics

Mixture volumetrics in U. S. practice are similar to European practice with the additional consideration of the VMA in the mixture. The level of acceptable VMA, however has not been established, and should not be the same as currently used for HMA mixtures. Current practices for compaction and density requirements are comparable to European practice.

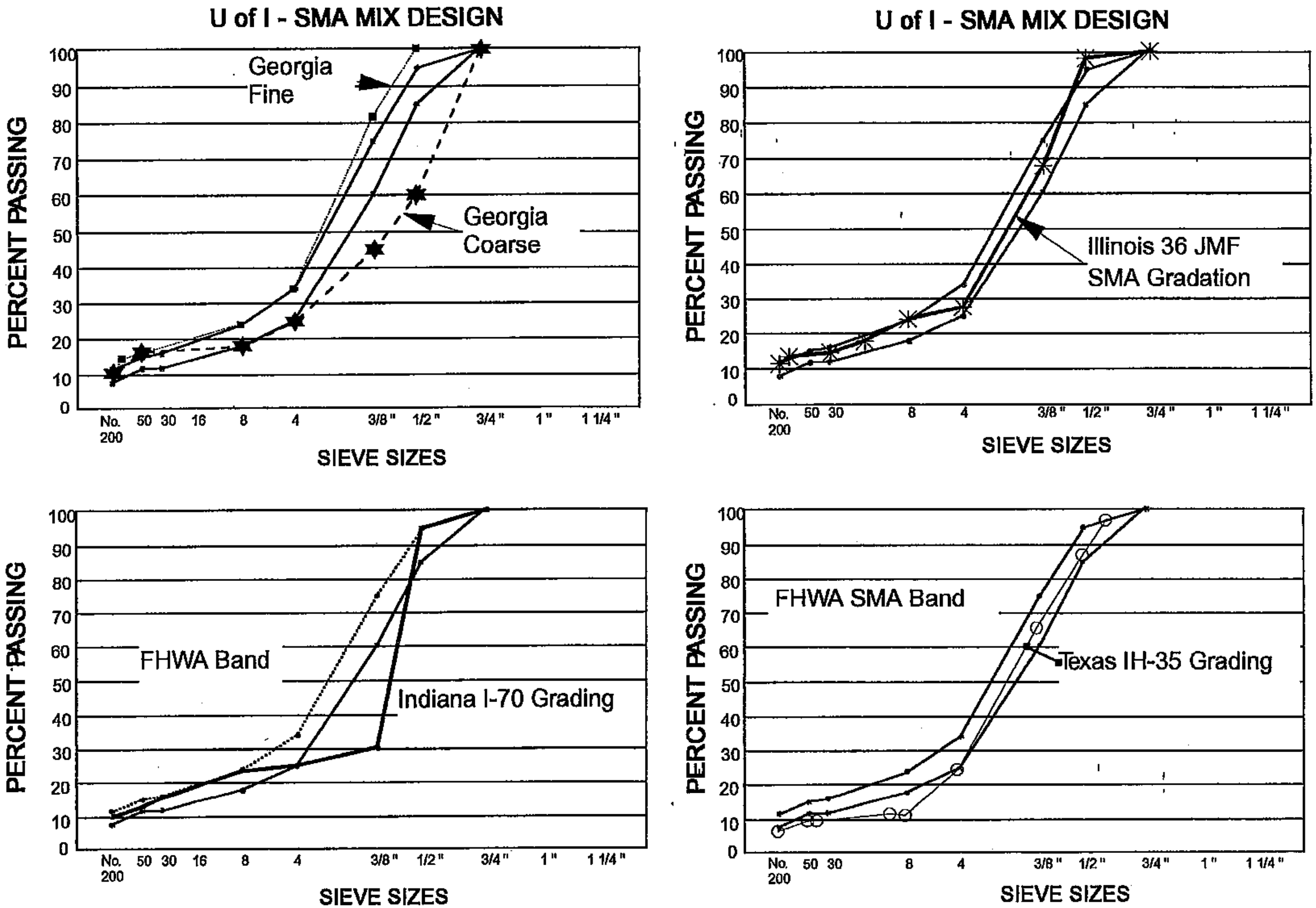

Figure 1-4. Various SMA gradations used in US construction.

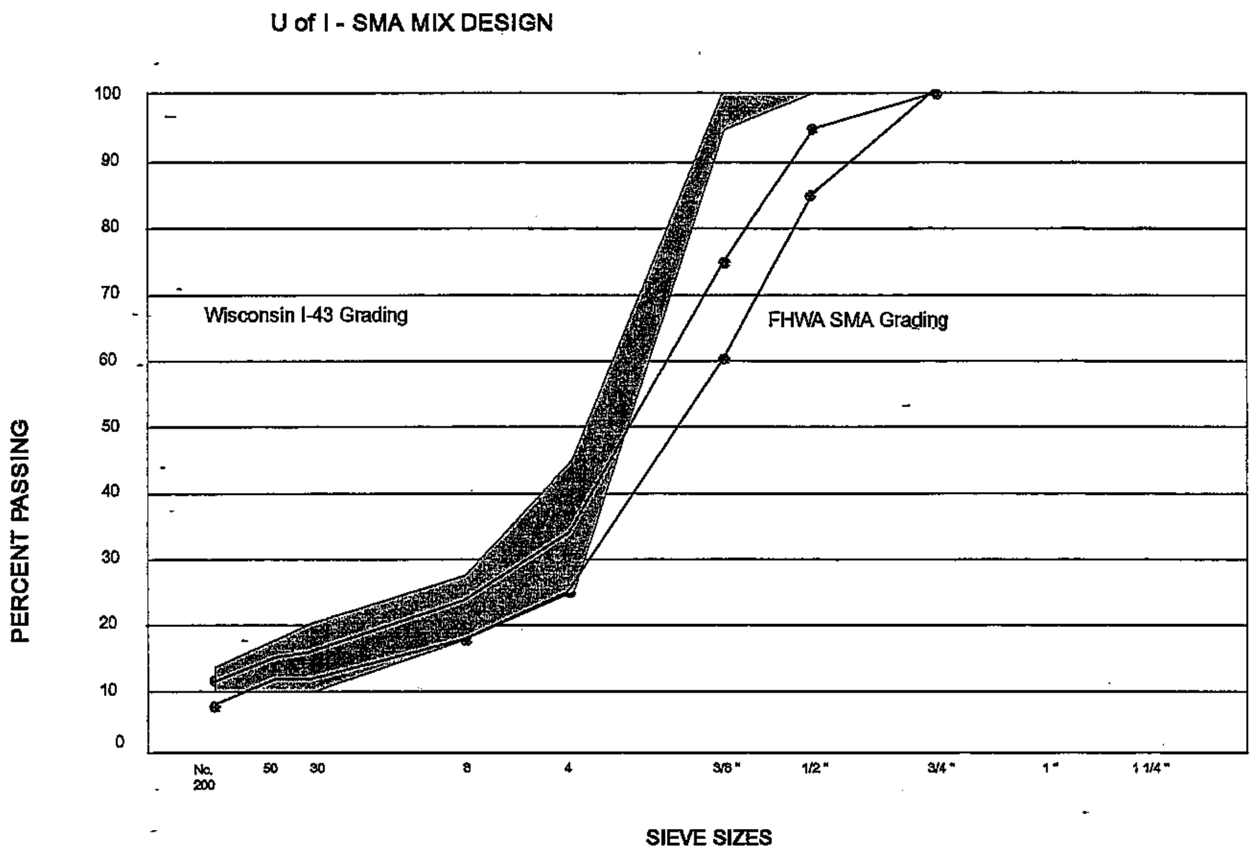

Figure 1-5. Wisconsin SMA gradation.

1.5 CURRENT STATUS OF SMA PROPERTIES

While the definition includes the important elements of an SMA mixture, it does not present the full extent of the complex nature of SMA mixtures, nor the difficulty commonly encountered in designing an SMA mixture to accomplish all the objectives included in this definition. Implicit in the definition of a true SMA mixture are the following:

- Asphalt rich
- Gap-graded
- Stone-on-stone skeleton
- Stabilized binder
- Sand/asphalt matrix
- Satisfactory mixture volumetrics

Difficulties arise when performing a mix design to produce a SMA mixture. First, the available mix design procedure for an SMA mixture is not formalized to the extent of the Marshall procedures. Second, the European experience provides no empetus for developing a uniform set of procedures, as each contractor develops proprietary processes, and keeps procedures somewhat hidden. A complete mix design procedure should ideally indicate a degree of compliance for each mix with some criteria which relate to a value that should be expected in a well designed the SMA mixture. At present there are no specific values which may be assigned to an SMA mixture to indicate compliance, and general values only are available.

- Asphalt Rich - It is presently recommended that 6 percent by total weight be used as a minimum acceptable value for asphalt content. There is discussion relative to this value, and it is not an accepted absolute although it does provide a target value that should be addressed and targeted.
- Gap Graded - The shape of the gradation curve emphasizes the concentration of stone sized particles, commonly the plus #4 sieve sized materials. This is a radical departure from the normal dense graded asphalt (HMA) gradations used by most DOTs. The precise nature of control on individual sieves under current US. practices may be different than European practices which may produce variations in gradations not seen in Europe on these mixes. If this variation produces properties which are not common to SMA mixtures, the gradation control will require further investigation.
- Stone on Stone Skeleton - The gap-graded gradation is necessary to provide a skeleton of stone particles which carry the traffic loadings. This philosophy is different from the HMA performance characteristics where all sizes carry the loads, even down through the sand sized particles. There must be some measurement of attainment of a stable stone on stone skeleton in the mixture before true SMA performance can be indicated.
- Stabilized Binder - Because there is a significantly higher amount of asphalt cement with a proportionately larger amount of stone sized particles, the film thicknesses on the aggregate particles is proportionately thicker than for normal HMA mixtures. This can cause draindown problems during mixing and placement producing problems with consistency. European practice calls for a fiber to be added at 0.3 or 0.4 percent by weight of mixture. This additive stabilizes the binder, reducing draindown from the mixture. There are questions over the effect of additive on binder rheology, and the use of polymers and specialty asphalt binders which have increased resistance to draindown, without the fibers.
- Sand-Asphalt Matrix - The stone skeleton must be filled, but not pushed apart, by the sand/asphalt matrix (minus #4 material). This proportioning could be vital to production of a stable skeleton, and for proper volumetrics in the SMA mixture.
- Satisfactory Mixture Volumetrics - Although the gradation is gap graded, the amount of sand sized particles and fines are increased to produce air voids that are between 3 to 4 percent. The precise value required for good performance

is being debated, but the important fact is that this gap-graded mixture does not have high air voids as is commonly found for these gradations. Current European practice does not stipulate Voids in the Mineral Aggregate (VMA), and does not even recommend calculating this quantity, which is felt to be extremely important to HMA mixture performance. Current discussions call for extremely high VMA values, near 19, for an SMA mixture

These quantities may or may not be necessary to produce a mixture of aggregate and asphalt binder that will behave in a similar manner to the SMA mixtures being produced in Europe. Very little structural testing information is available on the European mixes that would indicate acceptable levels. The suggested levels are felt to be representative of current SMAs constructed and performing satisfactorily in Europe. Thus, these values represent acceptable values, but are not substantiated by any design procedure or testing protocol that would allow testing to be performed on different materials to indicate a degree of SMA compliance. Given the different procedures and materials used in the US. such testing is certainly called for.

1.6 PROJECT FORMAT

The foregoing discussion illustrates the differences which should be investigated between U. S. and European practices. It also demonstrates that there is a significant amount of work required to establish precisely what properties are necessary to have an SMA mixture. It is clear that structural testing of these mixes will be required to indicate the relative effects of mixture composition, materials, additives, and what must be done to obtain the requisite properties of an SMA mixture.

To address the concerns expressed here, and to evaluate the ability of Illinois aggregates and asphalt binders to be successfully used in an SMA mixture this project was initiated by the Illinois Department of Transportation (IDOT) through the Illinois Transportation Research Center at SIU-Edwardsville (ITRC) with the Civil Engineering Department of the University of Illinois (U of I). The Objectives of this project included the following:

- Establish the state of the art in SMA mix design.
- Conduct mix designs using accepted materials available to IDOT.
- Determine structural properties of the mixes, and available field mixes to establish comparisons with standard IDOT HMAs.

- Highlight areas of importance in the SMA mix design and testing procedures, and where possible make specific recommendations on material suitability and testing procedures.

To accomplish these objectives, the approach selected utilized the following materials and combinations.

- Asphalt Cements
  - One AC-20 grade
  - One polymer modified asphalt
  - One fiber modifier
- Aggregates
  - Four diverse aggregate types
  - One mineral filler
- Gradation
  - FHWA recommended gradation
  - European gradation, German
- Accepted 50 blow Marshall mix design
- Structural testing at optimum asphalt content (VTM of 3 percent)
  - Diametral resilient modulus at 70, 90 °F
  - Indirect tensile strength at 20, 70 °F
  - Permanent deformation at 70, 95, and 120 °F
  - Creep modulus at 70, 90, and 120 °F
- Solicitation of field mixes for comparative testing
  - Wisconsin
  - Michigan
  - Missouri
  - Illinois

The remainder of this report presents the achievement of these objectives in the following chapters:

- Material Selection
- Mix Design and Sample Preparation
- SMA Mixture Verification Testing
- Structural Performance Testing
- Interpretation of Mix Design and Structural Testing
- Conclusions and Recommendations

2.0 MATERIAL SELECTION

2.1 INTRODUCTION

Material Selection on this project is important from the necessity of selecting a variety of materials that are typical of Illinois materials, while providing the potential of obtaining good SMA mixtures. The properties resulting in these SMA mixtures should be similar to the properties found in the European SMA mixtures. The important items in material selection include asphalt cement, additive type, aggregate type, gradation, and mix design procedure.

2.2 ASPHALT CEMENT

One grade of asphalt cement was chosen. An AC-20 from Emulsicoat, Urbana, Illinois was selected to represent the preferred grade of AC-20 for SMA mixtures. This grade of asphalt cement was used in all SMA and HMA mixtures. Additional samples of AC-5 and AC-20 were collected at the same time for use in the draindown investigation to illustrate effects of asphalt binder grade. A polymer modified asphalt cement from Elf Asphalt, Terra Haute, Indiana (now Koch Asphalt) was selected for use in the polymer modified mixtures. Figure 2-1 contains the viscosity temperature curves for the polymer modified and the AC-20 obtained using the Brookfield rotational viscometer.

Two cellulose fibers were obtained, one representing European fibers (Arbocel), and one representing domestic fibers (Central fibers). The domestic fiber was used in all mixes on this project calling for fiber addition. The European fiber was used for a comparison study of draindown and asphalt binder modification characteristics of the asphalts to demonstrate any differences which could be significant in attainment of SMA characteristics.

2.3 GRADATIONS

The number of different gradations that have been utilized in SMA construction to date were illustrated in Chapter 1. To maintain a uniform material, the FHWA recommended gradation, shown previously in Figure 1-2, is used for all SMA mixtures in this study. The German gradation shown previously in Figure 1-3 was selected for comparison purposes. These gradations will be used on select materials to develop material property differences between a typical European mixture, and the SMA mixtures prepared with Illinois materials. Traditional Illinois HMA mixtures will be prepared using the gradation shown previously in Figure 1-2. This gradation represents a high quality mixture typically called for in overlays of concrete Interstate pavements in the state. This mixture represents the most rut resistant mixture used by the Illinois DOT, and should provide a comparison to the, what should be superior, SMA rut resistance characteristics.

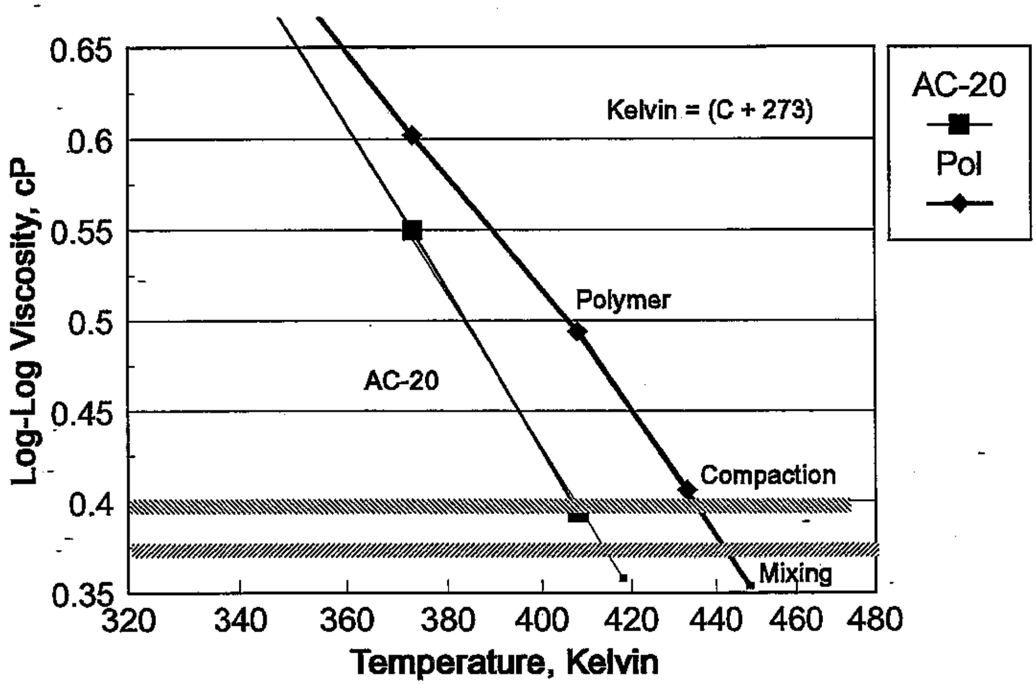

Figure 2-1. Viscosity-temperature curves for asphalt cements.

2.4 AGGREGATES

A variety of aggregates are used in HMA mixtures throughout the state of Illinois. Many of these represent aggregates which are felt by some not to be suitable for the SMA mixture. It will be beneficial to utilize aggregates representing a full spectrum of hardness to demonstrate differences resulting in the SMA mixtures. For this reason, the following coarse aggregates were selected:

- Limestone (LS) - Fairmount
- Dolomite (DO) - Lehigh
- Crushed Gravel (CG) - W. Lebanon
- Rhyolite (RH) - Iron Mountain

One fine (-#4 sieve) aggregate was used for all mixtures. The dolomite was used for this purpose. It is not felt that the matrix material plays a significant role in the SMA load carrying properties, and to emphasize the coarse aggregate impact on properties, the fine aggregates were kept the same in all mixes.

These aggregates represent a cross section of the softest material, the limestone, to a very hard durable aggregate in the rhyolite. The dolomite represents a harder carbonate than the limestone. The crushed gravel may provide an indication of SMA mixtures made with a siliceous composition.

These aggregates were obtained by the Illinois DOT, and transferred to the University of Illinois, meeting CA-11 and CA-16 gradations for the coarse aggregate, and FA-20 for the fine aggregate. Individual gradations were not determined for these materials as they were sieved to separate them into individual sizes of material for precise blending to meet the mixture gradation specifications. Individual sieve sizes included the following: 3/4 in, 1/2 in, 3/8 in, #4, #10, #40. The material passing the #40, retained on the #200 satisfied the gradation criteria for the mixes. The gravity properties of these aggregates are provided in Table 2-1.

Table 2-1. Specific gravity data of aggregates used in mix designs.

Aggregate	Bulk G_{sb}	Apparent G_{sa}
Fairmont (Limestone)	2.66	2.71
Lehigh (Dolomite)	2.69	2.77
W. Lebanon (Crushed Gravel)	2.68	2.75
Iron Mountain (Rhyolite)	2.66	2.67
Lehigh (-#4)	2.6	2.75
Mineral Filler	2.8	2.8
FA-20	2.6	2.75

The particle shape data for these aggregates is provided in Table 2-2. The Fairmont limestone had the largest percentage of particles with elongated or flat shapes, with approximately 30 percent being elongated. The remaining aggregates all had significantly fewer elongated particles. All aggregates would meet the current FHWA SMA recommendations for particle shape. It may be significant that the Fairmont limestone had approximately one-third of the particles with a cubical shape while the remaining aggregates all had half or more of the particles with a cubical shape.

Table 2-2. Particle Shape Distribution for Aggregates, +#4 Sizes.

	Ratio	Limestone (LS)			Dolomite (DO)			Crushed Gravel (CG)			Rhyolite (RH)		
Shape Factor	L/W	+1/2	+3/8	+#4	+1/2	+3/8	+#4	+1/2	+3/8	+#4	+1/2	+3/8	+#4
Cubical		33	39	31	56	53	53	60	57	43	54	54	43
Flat Round (Thick./Diam.)		27 (1:2)	18 (1:3)	19 (1:2)	23 (1:3)	22 (1:3)	27 (1:3)	23 (1:3)	20 (1:3)	36 (1:3)	18 (1:3)	24 (1:4)	27 (1:3)
Elongated Flat	<3	9	14	13	7	6	6	3	5	9	7	10	5
	>3	7	16	14	1	0	2	0	0	1	0	1	2
	>5	6	0	2	0	0	0	0	0	0	0	0	0
Elongated	<3	12	6	19	13	17	8	15	3	11	19	9	20
	>3	6	10	3	0	2	4	0	15	1	2	1	3
	>5	0	0	0	0	0	0	0	0	0	0	0	0

One mineral filler was used in all mixes. This filler was a fine limestone. The gradation properties and hydrometer analysis data are given in Table 2-3. This filler satisfies the concerns of not having too large an amount of material smaller than the 0.02 mm size in the final gradation. If this amount is greater than approximately 50 percent finer in the mineral filler, there could be problems with extending the asphalt cement.

Table 2-3. Mineral Filler Gradation

Size, mm	Percent Passing
#200	73
0.04	45
0.026	37
0.015	32
0.01	25
0.0074	18
0.0036	7
0.0015	3

2.5 TEST MATRIX

The mixtures to be analyzed on this project are shown in Table 2-4. This test matrix includes an Illinois DOT HMA gradation for all aggregates using the AC-20. The FHWA SMA gradation is used with all aggregates with the AC-20 and cellulose fibers. The German SMA gradation, with fibers, is to be used with the rhyolite, and the dolomite aggregates as they betterrepresent a more durable aggregate. The polymer modified asphalt is to be used with the FHWA SMA gradation and the rhyolite and dolomite aggregates.

Table 2-4. Sample Matrix composition for mixtures to be made.

Aggregate	IDOT	FHWA	GERMAN	POLYMER
Limestone	X	X		
Dolomite	X	X	X	X
Gravel	X	X		
Rhyolite	X	X	X	X

This combination of aggregates, gradations and asphalt binders provides for one on one comparisons between SMA and Illinois HMA mixtures for all aggregates. It provides for comparisons of European and Polymer SMA mixes with the FHWA recommended gradations using fibers. Comparisons will illustrate differences between current HMA mixtures and SMA mixtures typical of U. S. construction practices. The German gradation will illustrate whether the larger size of 3/8 inch material in this gradation produces a significant difference with the FHWA gradation.

2.5.1 Structural Testing

A mix design will be performed on each combination to establish optimum asphalt content. Sufficient samples will then be prepared at optimum asphalt content to allow structural testing to be performed to develop sufficient data to indicate the differences which may exist between the different mixtures. The following tests, which will be explained in more detail in a subsequent chapter will be performed:

- Diametral resilient modulus at 70, and 90 F will establish the modulus of each mixture and indicate the degree of temperature sensitivity which may be different in mixes with fibers.
- Indirect tensile strength tests will be performed at 20 and 70 F. The testing at 20 F will indicate any low-temperature differences which may arise from the different binders used. The strength at 70 F is an indicator of quality levels, and provides some indication of fatigue resistance, which is not felt to be a distress that occurs in SMA mixtures which are typically used only in the top 2 inches of the pavement.
- Permanent deformation (repeated loading) testing will be performed at 70, 95, and 120 F to indicate any significant differences in rutting resistance that may be exhibited by the different mixes.
- Creep Relaxation testing will be performed at 70, 95, and 120 F to provide supporting data for the permanent deformation testing, and for modulus comparisons in compression testing, which may be significantly different from diametral values for SMA mixtures.

This testing will illustrate structural differences between four different aggregates when used in similar mixtures, IDOT HMA, and FHWA SMA. Two aggregates will be tested to allow comparison of the IDOT and FHWA gradations with a European gradation to illustrate any benefit which may be obtained from the more traditional gradation. The use of a polymer modified asphalt binder in the same two aggregates allows evaluation of the impact of a different method of stabilizing the asphalt. While actual performance comparisons will not be performed, these structural properties will indicate if significantly different mixture behavior can be anticipated for any of the material combinations. Further, the properties of the SMA mixtures will indicate if they are suitable replacements for the upper 2 inches of the asphalt pavement structure.

2.6 FIELD SAMPLES

Contact was made with several Highway agencies that had placed SMA mixtures within the previous year to request samples of mixture or cores. The Wisconsin DOT provided an extensive number of cores from their SMA projects. The gradation was previously shown in Figure 1-5. The different materials used in each mixture and paving layers are as follows:

- Section 1 - Vestoplast - 10 %
- Section 2 - MAC - 20
- Section 3 - MAC - 10
- Section 4 Control, Wisconsin mix
- Section 5 - Organic Fibers
- Section 7 - Inorganic Fibers
- Section 8 - Vestoplast - 5%

The Illinois DOT provided bag mixes from two recent SMA projects which were reheated in the laboratory and compacted in the Marshall hammer at 50 blows. The gradations for these two mixes are shown in Figure 2-2. Testing on these samples will allow a comparison between laboratory controlled mixes and field mixes taken from the truck and cores from the pavement. Significant differences could indicate that more consideration must be given to the mix design procedure, and perhaps require design to be done with more reliance on field samples or lab samples. Lack of differences could provide support that laboratory samples provide an indication of field behavior for the SMA mixtures.

2.7 SUMMARY

Materials were selected that have the potential to produce acceptable SMA and HMA mixtures. They were chosen to provide a variety of properties which could interact to produce different materials in the final mixture. The intent of this project is to characterize the different mixtures produced and assign material property differences with the mixture property differences which may be noted. This information will provide the basis for altering any mix design procedure to include important material property considerations which may not be adequately recognized at present.

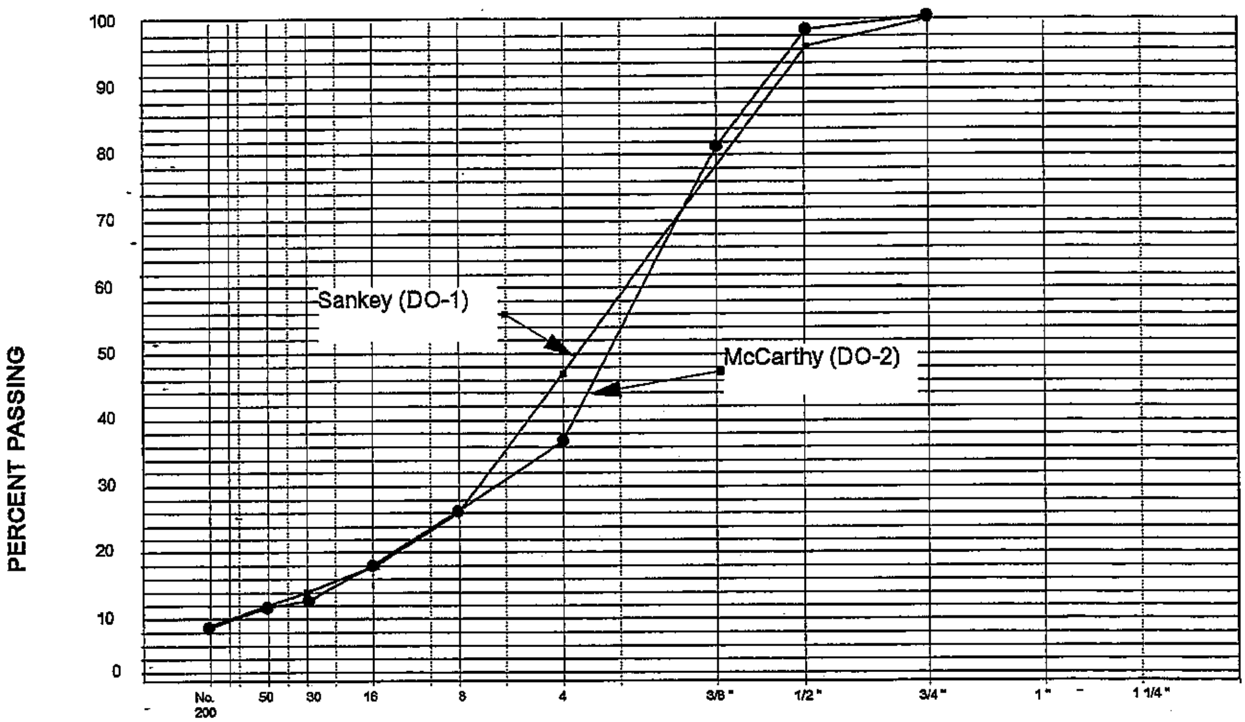

Figure. 2-2. SMA gradations constructed by IDOT.

3.0 MIX DESIGN AND SAMPLE PREPARATION

3.1 MIX DESIGN

Mix design is generically a combination of aggregate and asphalt cement in a process that produces samples representative of that which would result on the pavement and be subjected to traffic. The material combinations recommended from the mix design procedure should be workable and attainable in the field with normal handling procedures. The mix design process should also provide a means of evaluating the various performance characteristics of the mixture. Because the SMA mixture contains significantly more components that could impact performance, the mix design for these mixtures can be quite a bit more complicated than a traditional mix design for a dense graded hot mix asphalt (HMA). The criteria that have been used in the past cannot be applied directly to the SMA mixtures, although the general design procedure may still be applicable when structured properly. This chapter presents the mix designs for the IDOT and SMA gradations with the various material combinations. Preliminary information concerning suitability of materials for use as an SMA mixture are also provided to indicate concerns in SMA property development.

3.2 DENSE GRADED HMA MIX DESIGN

The dense graded HMA mix design procedure generally follows the Asphalt Institute procedure in MS-2[5]. Mixing and compaction temperatures were taken from the viscosity temperature curves shown earlier for the AC-20 asphalt cement. Samples to be compacted were prepared using aggregates separated into the individual sieve sizes. Amounts of each sieve size were added to the sample pan to produce a final gradation matching the center of the envelope shown previously in Figure 1-2. Three samples were prepared for each of five asphalt contents. One sample was prepared for sieve analysis to check gradation compliance and one sample was prepared for a maximum density determination.

These aggregate samples were placed in an oven overnight at 325 °F. The asphalt cement was heated to the mixing temperature the following morning, and the appropriate amounts of asphalt cement were added to the aggregate and the sample was mixed. The mixed sample was placed back into the oven to bring it to the compaction temperature (less than 20 minutes). Each sample was compacted at the appropriate compaction temperature using a Marshall hammer with 75 blows per side. The samples were allowed to cool overnight before extruding and testing. A sieve analysis was performed on the selected sample, and the theoretical maximum density was determined on the selected sample.

3.2.1 Mix Design Results

The SMA mix design procedure using 50 blow Marshall compaction was performed. Included in the analysis are density, VTM, VMA, VFB, Stability, and Flow as a function of asphalt content. The data used were the average of three samples tested at each asphalt content. Selection of optimum asphalt content was made to provide 4 percent air voids, with satisfactory VMA, Stability, and Flow. Table 3-1 provides the mix design data for each aggregate which will be used for sample preparation.

Table 3-1. Mix design results.

Aggregate Type	Mix Value	SMA Fibers	SMA Polymer	SMA German	IDOT HMA
Limestone	P_b, %	6.1	--	--	5.5
	VTM	4.3	--	--	4
	VMA	17.8	--	--	16.5
Dolomite	P_b, %	6.4	6.5	5.8	5.8
	VTM	3	3.8	4.5	4.5
	VMA	17.3	18	17	17.5
Crushed Gravel	P_b, %	6.2	--	--	5.8
	VTM	5.2	--	--	3
	VMA	19	--	--	16.3
Rhyolite	P_b, %	6.4	6.7	5.8	6
	VTM	3	3	3	4.5
	VMA	16.8	19	16	18

3.3 SMA MIX DESIGN

The SMA mixture design procedure generally followed the FHWA model specification. As mentioned previously, the FHWA gradation was selected as the target gradation for all samples, with two samples being blended to the German gradation. The crucial items which should be investigated before or during an SMA mix design to ensure a suitable SMA mixture is possible include:

- Draindown Characteristics
- Stone/Sand Compatibility
- Volumetrics

The normal progression of the mix design is to first select a mix gradation and blend the available aggregates to satisfy the specification, and then evaluate the compatibility between the stone and sand sizes in the gradation. When these proportions are deemed appropriate to produce an SMA mixture, the mix design process is very similar to the standard process. Various amounts of asphalt cements are mixed with the blended aggregates. The main difference is that a minimum 0.3 percent of fibers, by total weight of mix, are added before mixing to stabilize the asphalt cement against draindown. The samples are mixed and compacted at 50 blows per side. The optimum asphalt content is selected to satisfy void quantities. Marshall stability and flow testing are performed, although at present only recommendations for Marshall stability and flow are provided().

Following selection of optimum asphalt content, the mixture is evaluated for draindown characteristics. Current recommendations follow the Schellenberg beaker test, but a procedure under development by NCAT may be more appropriate when additives other than fibers are being used. If the draindown of asphalt binder exceeds a percentage of original mixture weight, the asphalt content should be reduced, or the amount of additive increased. These two adjustment procedures both have drawbacks. Reduced asphalt content will increase air voids, and should be done judiciously. Changing the amount of additive requires a new mix design to validate the voids in the final mixture, requiring more work in the lab, but providing a more acceptable mixture to use with normal field variability.

These individual components are discussed here to present the results found, and the material properties noted during the mixing process and to establish the basis for further investigation.

3.3.1 Stone - Sand compatibility

The purpose of the SMA gradation is to provide a stone structure with the +#4 material filled by the sand asphalt matrix made from the -#4 material. The specific sieve used to make this separation may change with different gradations, but the #4 sieve appears adequate for the materials used in this investigation.

Using the blended aggregate gradations, the voids in the +#4 material are typically established by compacting the aggregate to a refusal density. This was done using a vibratory hammer drill. The voids in the -#4 material are established by rodding the material in differing amounts of effort to produce a range of bulk densities. These densities are given in Table 3-2 and Table 3-3 for various densification conditions.

Table 3-2. Densities for coarse aggregate portions (+#4), pcf.

Aggregate Type	IDOT HMA	FHWA/SMA	German SMA
Limestone	105.3	105.9	106.3
Dolomite	111.1	109.5	108.7
Crushed Gravel	106.5	107.2	105.1
Rhyolite	105.6	104.1	104.2

Table 3-3. Densities for fine aggregates under various densification procedures, pcf.

Compaction	IDOT	FHWA/SMA	German
None	113.2	121.4	116.7
Light Rodding	119.4	130.7	128.8
Light Rodding Plus Tamping	124.3	135.5	132.7
Vibratory Hammer	129	149.3	148.6

The compacted density of these fractions is used with the bulk specific gravity given earlier to calculate the void space remaining in the aggregate as it is compacted. The void space in the +#4 material is occupied by the -#4 material and the asphalt cement, the sand/asphalt matrix. Thus, the proportions of stone and sand can be approximately calculated using rodded densities and specific gravities. The calculated proportions using the aggregate density and specific gravity values for both the SMA and IDOT HMA gradations to produce a VMA of 17 percent was very similar for all, with 74 percent required for the +#4 aggregate, and 26 percent required for the finer aggregate. The relative proportions of these aggregates to maintain three percent air voids is near the preferred 30 percent passing the +4 sieve, but slightly on the low side. The IDOT HMA gradation required slightly less coarse aggregate, but still very close to the desired ratios. These values indicate that strict adherence to the 70/30 ratio may not be best practice.

The closeness to the desired ratio indicates that the SMA gradation used here in combination with the aggregate types should produce a mixture with reasonable compatibility, resulting in stone on stone contact. This initial comparison, however, should not be accepted in lieu of testing the final mixture. These tests on aggregates are only a preliminary indication of compatibility, and a structural test of the compacted mixture is required for a conclusive indication of stone skeleton. Comparisons such as these are useful as they can show a marked difference in proportions required to achieve a preset level of voids in the mixture. A preliminary evaluation of the aggregate testing presented here would be that a slight increase of the proportion of +#4 sieve size material might be preferable to the accepted standard 70 percent level of the FHWA gradation.

The degree of compatibility for the IDOT HMA gradations indicates that the current IDOT mixes generate a great deal of stone interlock separate from the sand sizes present. The sand can be expected to produce a matrix filling the stone structure somewhat, although not nearly as evident as in the SMA gradation. These mixes could be expected to exhibit a high resistance to deformation as a result of the aggregate stone structure.

The gradations and materials proposed for use on this project appear to offer a high degree of compatibility between the stone and sand sizes for the aggregates used here. It is certainly possible that different aggregate types with different particle shape distributions and surface textures would produce a different degree of compatibility, even following the FHWA gradation. For this reason, every aggregate blend proposed for use in an SMA mixture should be investigated in the manner similar to that illustrated here to generate an indication of compatibility. Gradations that accurately represent the gradations that will result in the field should be used to develop these indications.

3.3.2 Draindown Testing

The draindown characteristics of the SMA fiber mixtures were evaluated using the Schellenberg beaker procedure. This procedure uses the total SMA mixture. The mixture is placed in a glass beaker and placed in an oven at 160 °C for one hour. Following the heating period, the beaker is removed from the oven and inverted to remove the mixture. The asphalt clinging to the beaker represents the asphalt binder that has drained out of the mixture. The larger this amount, the more likely that problems will develop during mixing and transport of the SMA mixture. The recommended maximum level for SMA mixtures with fibers is less than 0.3 percent binder by total weight. The results of testing for the aggregates in this study were consistently at or below 0.1 percent. All mixes here would be acceptable from the draindown results as shown by the beaker procedure with the minimum of 0.3 percent fibers.

3.3.3 Mix Design Results

With draindown and sand/stone compatibility being indicated as satisfactory, the mix designs for the different mixtures were performed. The samples were all mixed and compacted at 50 blows per side at 300 °F to 325 °F temperatures to maintain satisfactory viscosity requirements, although the viscosities varied with the different amounts of fibers required for the different asphalt contents. Appendix B contains the mix design curves for the SMA mixes.

These SMA mixtures can all be evaluated to select an optimum asphalt content for a VTM of 3 percent, which was the criteria in effect during this stage of the project. The mix design values shown in Table 3-1 are representative and would indicate a reasonable SMA mixture has been obtained. However, it is likely that these are not necessarily an optimum mixture composition. VMA values below 17 would indicate that the gradation should be altered to increase the voids in the mixture. This would appear to be necessary for the aggregate types used here for several of the mixes, but was not done as the purpose of this study was to investigate the effect of the different aggregates on mixture properties resulting from the gradation and material differences.

3.4 SUMMARY

The asphalt contents determined in the mix design testing presented in thie chapter were used to prepare a series of 12 samples of each mixture to be compared in the structural testing phase of the project. The data generated during the SMA mix design process indicated the need for a more thorough evaluation of draindown and stone skeleton characteristics. These are discussed in detail in the next chapter.

4.0 SMA MIXTURE PROPERTY VERIFICATION

4.1 INTRODUCTION

The mix designs presented in the previous chapter were performed using a set gradation and specific aggregate types. This was done to indicate differences which could be obtained with the different materials that might indicate where improvements should/could be made to obtain a more representative SMA mixture. This chapter will present testing that was performed on the materials and mixtures to indicate the extent to which SMA mixture properties have been achieved, and what must be done to verify the attainment of these properties in a mix design procedure.

Items investigated include some material properties of the asphalt and aggregate that have been described as a prelude to performing the mix design, and will not be discussed here in detail, but only as they pertain to providing insight into the nature of the SMA properties resulting in the mixture. Important SMA intrinsic properties include:

- Binder Stabilization
- Proper Proportioning of Sand and Stone
- Verification of Stone on Stone Skeleton
- Volumetrics of Components
- Particle Shape Distribution

4.2 ASPHALT BINDER STABILIZATION

4.2.1 Draindown

All of the mixtures examined here met the requirement of draindown for the Schellenberg beaker test, of 0.3 percent draindown after one hour at 160 °C with the recommended dosage of 0.3 percent by weight of fibers. These results were shown in Table 3-4. It was felt during mixing and subsequent testing that there were differences between the aggregate types. This will be shown in later sections. It was felt that a more sensitive draindown test was needed to more clearly indicate the interaction of stabilizer, asphalt, and aggregate type.

Small aggregate particles exert a more significant attraction to asphalt cement, and hold it in thinner films than the larger particles. They have a more overriding effect of asphalt mobility, and increasing the amount of fines can significantly decrease the amount of draindown, thus making variability in production a critical consideration. For this reason, the larger particle sizes more directly interact with the stabilized binder, and should be more sensitive to asphalt property changes. This was the basis for examining the +#4 sieve aggregates separately to develop an indication of additive effects on draindown.

The test equipment was changed to a flat plate to de-emphasize the structural effect of aggregates impeding downward flow of the binder. The mixture gradation for the +#4 material was the proposed gradation. The time and temperature were maintained at the same level proposed in the beaker test (160 °C and one hour).

4.2.2 SMA Gradation Results

The standard FHWA gradation was used to prepare samples of +#4 sieve sized material. Two samples were prepared for each test result. Figure 4-1 contains the results for the flat plate draindown tests. It is clear that there is a definite aggregate difference in this test for the coarse aggregates, as indicated with the flat plate configuration. The limestone materials clearly had less draindown than the crushed gravel and rhyolite in this test configuration. This difference was not obvious from the beaker evaluation procedure. If the flat-plate procedure had been used to adjust asphalt draindown properties, the amount of fiber would have been increased for these two mixes. It is apparent that the stabilization does not completely stabilize all the asphalt cement, and the interface binder may still be sensitive to aggregate type.

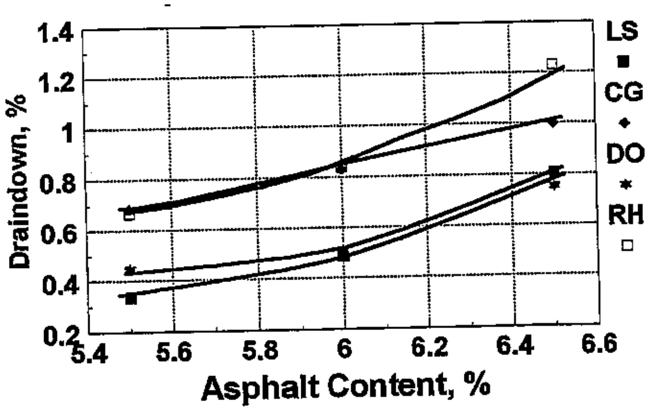

Figure 4-1. Flat Plate Draindown results for four aggregate types, SMA gradation.

4.2.3 IDOT HMA Gradation Results

The IDOT HMA gradations were evaluated using the unstabilized AC-20 asphalt cement. This testing provides a base line comparison value indicating what draindown characteristic currently is found in standard HMA production. Figure 4-2 contains the flat plate draindown data for the unstabilized AC-20. These results are only slightly higher than the fiber stabilized results from the SMA gradations with crushed gravel, when comparison is made at the optimum asphalt content for each mixture. This indicates that stabilized and nonstabilized binders in thick (SMA) and thin (HMA) films may have somewhat similar draindown characteristics. The SMA optimums are near 6 percent while the IDOT HMA optimums are near 4.75 percent, producing very different film thicknesses.

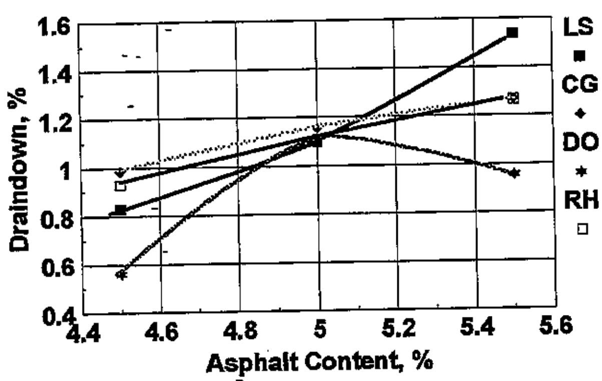

Figure 4-2. Draindown results for unstabilized AC-20, IDOT HMA gradations.

These comparisons indicate that there is little difference between aggregate types for unstabilized binders. However, stabilized binders do show some effect of aggregate type on the draindown characteristics, and the crushed gravel interface requires more consideration for stabilizing the asphalt binder. Fiber addition may not be the preferred method with these materials. Polymer modification which changes the whole binder may be more successful in eliminating aggregate type interface effects which appear to be acting here.

4.2.4 European vs. Domestic Fibers

The effect of variable fiber contents is shown in Figure 4-3 which examines draindown for the limestone aggregate at different asphalt contents with three different fiber concentrations of 0, 4, and 6 percent by weight of asphalt. Typical doses range in the 5 to 6 percent range for cellulose fibers. Also shown in this figure is a comparison between the European and Domestic fibers at the 4 percent dose rate. The testing indicate only a marginal difference in draindown characteristics, with the domestic fibers stabilizing the asphalt binder slightly more adequately.

4.2.5 Asphalt Rheology

The requirement to alter the amount of fiber additive changes not only the draindown characteristics of the asphalt cement binder but also the rheological properties of the binder. This is shown in Figure 4-4 which show the effect on Penetration and Ring and Ball Softening point for three grades of asphalt cement with different amounts of European cellulose fibers. The fiber percentages here are calculated as a weight of binder. There is little difference between the fiber types for the softening point, with the Domestic fibers showing a slight increase in softening point. There is a more observable difference in penetrations. The European fiber appears to decrease the penetration slightly more than the Domestic fibers. Because draindown is critical at the elevated mixing and transport temperatures, the softening point may be more indicative of draindown performance, and the draindown tests of Figure 4-4 indicates the Domestic fibers to do a slightly better job of stabilizing the asphalt binder as indicated by the rheology comparison with the draindown. results.

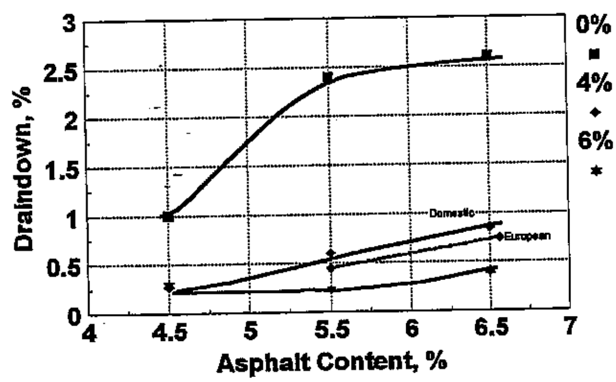

Figure 4-3. Draindown on limestone with different fiber doses.

It remains to be seen if a standard level of rheological modification can be developed rather than the inaccurate draindown test. It would be preferable to have a standard level of softening point and perhaps Penetration Index (PI) or Penetration-Viscosity Number (PVN). This would provide for a direct comparison of all binders. Polymer modified asphalts and specialty asphalt cements could be directly compared using these standards. Some modifications could be required for aggregate type effects.

4.3 SAND/STONE PROPORTIONS

The aggregate investigation in the previous chapter indicated that the relative proportions of sand and stone were at least approximately compatible, and could be expected to produce an acceptable SMA aggregate. This was born out in the mix design where reasonable mixtures were produced, but where some fine tuning was indicated to increase the level of VMA in the mixtures. The main method of achieving higher VMA values is not to decrease the asphalt content, but to adjust the sand/stone proportions.

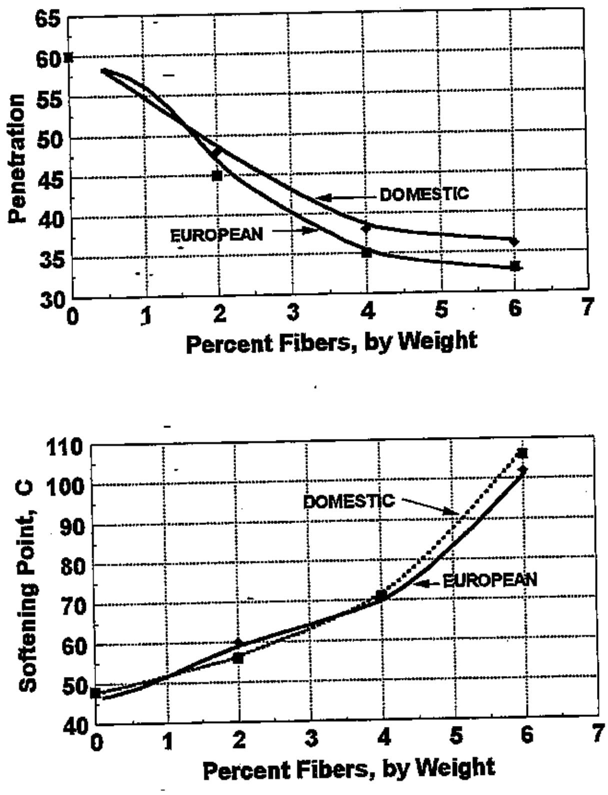

Figure 4-4. Asphalt rheology changes with fiber content.

To demonstrate the effect of varying the sand/stone proportions, the Fairmont limestone aggregate was selected for a parameter study in which the relative proportion of these two size fractions was varied. The ratios investigated included 35/65, 32/68, 30/70, and 25/75 to represent values that would cover good to bad proportions for normal usage.

These samples were prepared and a mix design was performed in the same manner as was done for the FHWA-SMA mixtures. The VMA and VTM curves for these different mixtures are given in Figure 4-5 and 4-6. It is clear that increasing the proportion of sand in the blend decreases the VMA considerably. Increased VMA values, mentioned as highly desirable for these mixtures, could be obtained by increasing the stone proportion in the gradation. It can be seen that more appropriate VMA values for the limestone gradation used here can be obtained with very little alteration to the gradation. The values used here do not alter the gradation enough to violate the proposed gradation envelopes. The resulting gradations for these samples are shown in Figure 4-7. Structurally there is very little difference in these mixes, as indicated by the diametral resilient modulus test data shown in Figure 4-8.

These data would indicate that improvement in the volumetrics by increasing the stone content and increasing the VMA can be done quite effectively without sacrificing mixture integrity, and should be examined when volumetrics indicate that the stone skeleton is being overfilled or underfilled. The volumetric data for the limestone mixture investigated here did not really indicate serious overfilling of the stone skeleton, and the adjustments shown here are relatively minor. The other mixes could have benefited from some adjustment in the sand/stone proportions to provide better volumetrics

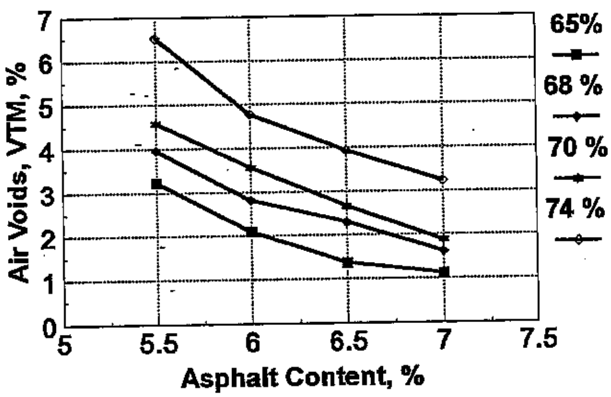

Figure 4-5. Change in VTM with stone content.

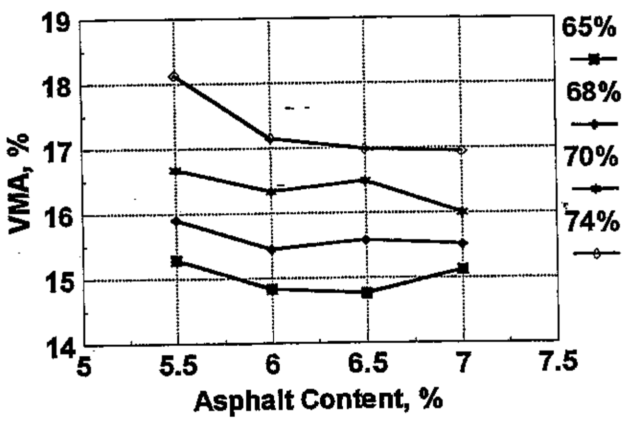

Figure 4-6. Change in VMA with stone content

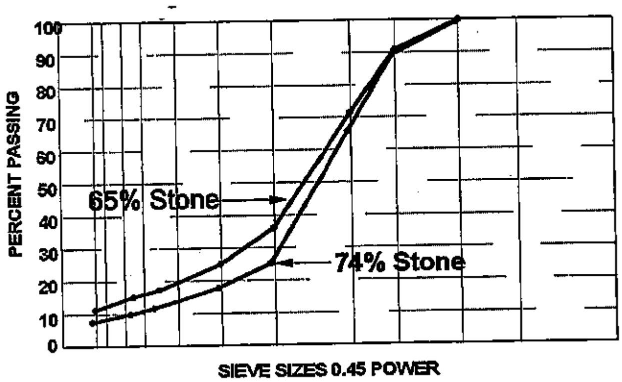

Figure 4-7. SMA gradations with high and low stone content.

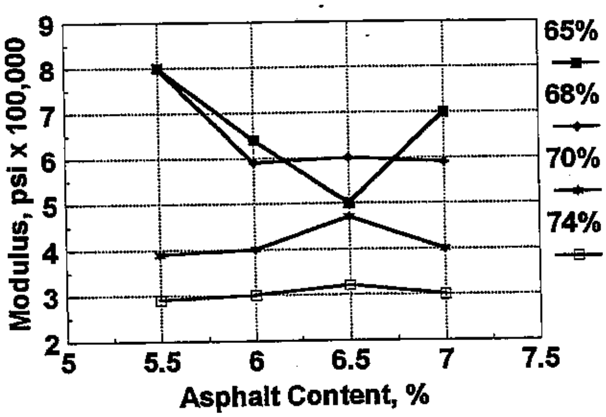

Figure 4-8. Diametral resilient modulus for various stone contents.

4.4 STABLE STONE ON STONE SKELETON

The determination of compatibility and volumetrics clearly indicates that a valid combination of materials and gradations can be achieved before and during the mix design. These properties do not guarantee that the criteria for an SMA mixture has been achieved, namely the development of a stable stone on stone skeleton to carry the weight of the traffic. They provide a good indication that this can be achieved, but further testing is required for verification.

To illustrate the degree of stone on stone lockup that was achieved in the proposed mix designs, samples were prepared and mixed at optimum asphalt content for both the SMA and IDOT HMA gradations in the exact same manner as was done for the mix design. Multiple samples were compacted at varying compactive efforts to illustrate the change in density resulting from increased compaction effort. A stable stone skeleton in the mix at 50 blows should not densify under increased compaction. Increased blows of 75 and 100 blows per side were used to illustrate the degree of stability in the designed SMA mixtures.

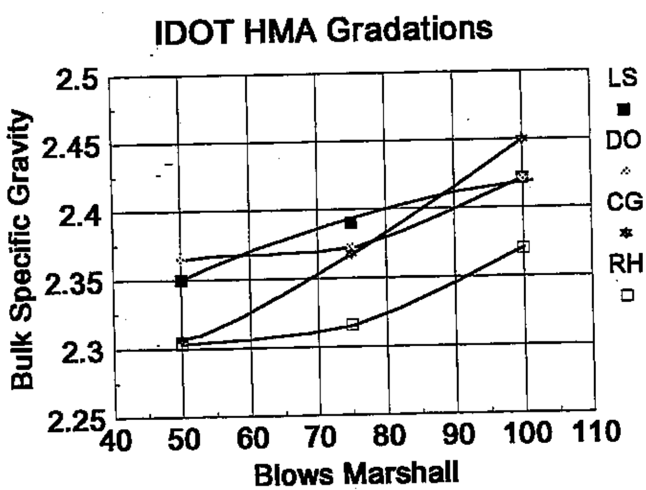

Figure 4-9. Change in density for the IDOT HMA gradations with different compaction efforts.

Figure 4-9 contains the results of the compaction study for the IDOT HMA gradations. It must be recognized that these mixtures were designed using 75 blows. There is a distinct trend even at the higher compactive efforts indicating that the mixture is continuing to be compacted in the expected manner for a dense graded HMA. While the degree of density loss in these IDOT HMA gradations may not be as high as is published for more traditional dense graded mixtures, indicating that these mixtures have a relatively higher degree of internal stability, there is no distinct locking of the aggregate matrix evident from 75 to 100 blows.

Figure 4-10 contains the results of the compaction study for the SMA gradation mixtures. These mixtures were designed for 50 blows. There is significant compaction evident when going from 50 to 75 blows per side, similar to what was observed for the

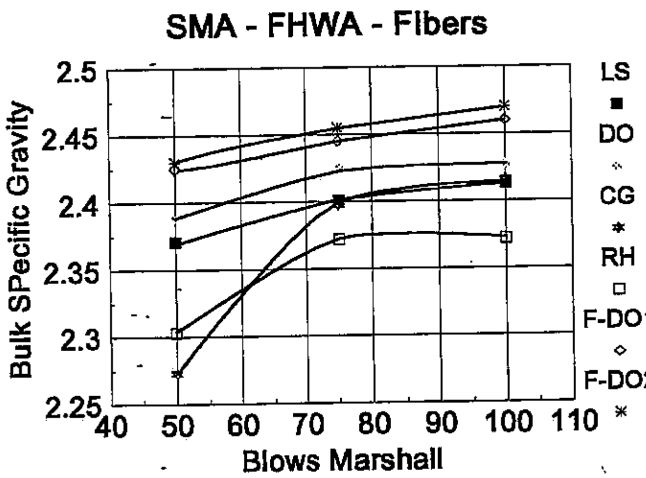

Figure 4-10. Densification results under different compaction efforts for the FHWA SMA gradation with fibers.

HMA when going from 75 to 100 blows. However, there is little evidence of continued compaction when going to the higher compaction effort of 100 blows. It appears from this data that 50 blows per side may not be sufficient to truly develop the stable stone skeleton expected in an SMA mixture using the aggregates in this study, and the FHWA gradation. The limestone aggregate developed the most interlock at 50 blows per side, while the others did not develop a similar degree of interlock until 75 blows per side compaction was used. The aggregate type did not seem to relate to this phenomenon, but the particle shapes may be important here as all aggregates were similar except for the limestone.

The limestone has a lower amount of cubical particles. This compaction study may provide an initial indication that some degree of elongated particles may be useful in producing an interlocked stone structure under 50 blow compaction in the laboratory, but that highly cubical particles may require more compaction to tightly orient them and produce an interlocked structure.

4.5 SUMMARY

The information presented in this chapter illustrates some of the considerations that should be investigated during a mix design to ensure an SMA mixture is being developed. The information also points out some of the items that can be used to define the nature of a mixture to classify it as an SMA mixture.

The limited testing calls into question the adequacy of a standard 50 blow Marshall compaction effort, and would indicate the need for each aggregate blend to be evaluated to determine if indeed there was an interlocked stable structure being developed in that particular mixture. The reliance solely on gradation compliance would not appear to be a good choice, and individual testing should be performed to verify SMA compliance.

The stabilization of the binder should not be done without consideration of the aggregate type being used in the SMA mixture. Draindown testing requires further improvements, as initial testing indicates that the same binder with the same fiber will perform differently on different aggregate types. Current procedures do not separate this performance adequately to allow additive percentages to be established for different situations.

5.0 STRUCTURAL CHARACTERIZATION OF SMA MIXTURES

5.1 INTRODUCTION

The mix design and sample preparation presented in previous chapters indicates that adequate mixtures can be prepared using the materials and gradations specified for this project, even though some fine tuning would be required in practice before a final mix design was approved. The main purpose of this project was to produce mixes with the specified materials, and compare structural characteristics of the resulting mixtures to allow qualitative statements about performance potential to be made. This chapter presents the results of the structural testing program.

The structural testing included the following:

- Diametral resilient modulus at various temperatures to provide a comparison of deformation characteristics under load that would be of use in structural design program.
- Indirect tensile strength at high and low temperatures to indicate relative quality of the mixtures and an indication of low temperature cracking differences.
- Repeated load permanent deformation testing at normal and elevated temperatures to indicate differences in rutting resistance.

The following sections will present the data collected on the individual mixtures, and for the specialty studies discussed in Chapter 4. In most cases two sets of data will be presented for discussion, one for the mixtures at various asphalt contents used for the Marshall mix design, and one for the samples prepared as optimum asphalt content. The optimum asphalt content data represents an average of three or four individual samples, while the test data for various asphalt contents will be derived from two samples, either of which could have been tested in another procedure or at another temperature. The optimum samples were typically tested only once prior to compressive creep testing. The samples for testing at optimum asphalt content were grouped according to density, and a cross section were placed in each testing group to have high, medium, and low air voids (within the range resulting when one asphalt content is used) represented in the testing.

5.2 DIAMETRAL RESILIENT MODULUS

The resilient modulus tests were conducted on the Marshall mix design samples prior to stability testing. Tests were also conducted on the samples prepared at optimum asphalt content determined in Chapter 3. This testing provides comparative data for both optimum asphalt conditions, and for the variation in modulus at other asphalt contents which could be used as comparison of structural differences resulting with asphalt content variations. It is important to remember that this test is primarily a binder test, and not an aggregate interlock test. Because it is a tension test, the importance of binder consistency and film thicknesses is perhaps overemphasized by this test configuration, relative to the influence of aggregate characteristics.

5.2.1 Individual Mixtures

The plots of resilient modulus at 70 F for the individual mixtures are shown in Figures 5-1 through 5-4. The change in resilient modulus as a function of asphalt content provides an indication of the structure of the mixture that is impacted by the asphalt content. Figure 5-1 presents the results of the IDOT HMA mixtures, which is considered typical of dense graded mixtures. The IDOT HMA gradations, as mentioned previously, are slightly more oriented to a stone on stone contact than more traditional dense graded HMA mixtures and the normally expected peak in the curve is not quite as evident.

The SMA gradations with fiber additions are shown in Figure 5-2. The trend in the data is not as distinct for the SMA gradation as it was for the dense graded mixtures. This indicates a more consistent utilization of the asphalt binder in relation to the aggregate in the mixture. With the dense graded mixtures, as the asphalt content changes, the influence of the binder on the structural characteristic of the mixture also changes, while for an SMA the asphalt content does not play as important a role in changing the structural characteristics of the mixture.

The SMA mixtures with polymer binder are shown in Figure 5-3, and the German SMA gradation results are shown in Figure 5-4. The trends are similar for these mixtures,

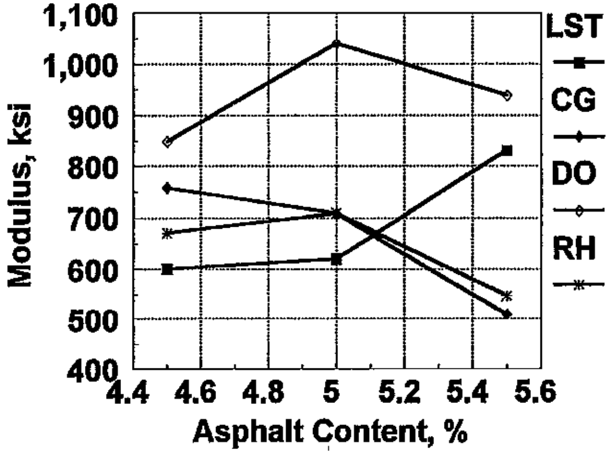

Figure 5-1. Variation in diametral resilient modulus for IDOT HMA gradations mixes.

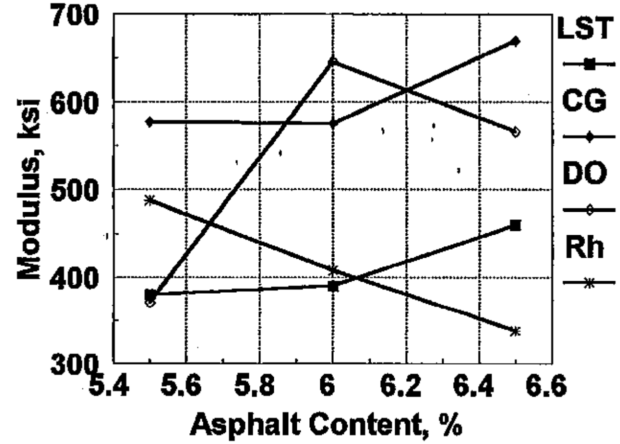

Figure 5-2. Variation in resilient modulus for FHWA SMA gradation mixes.

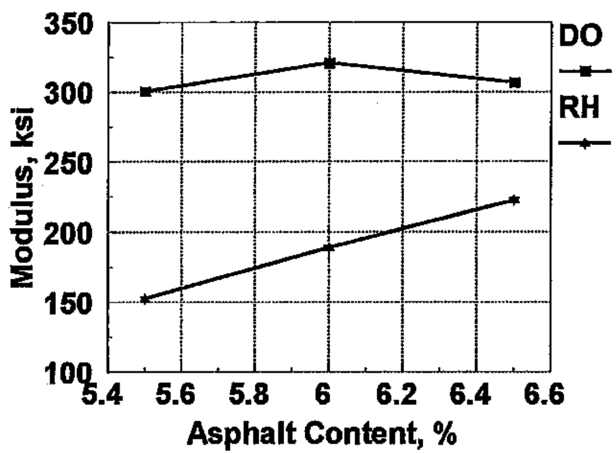

Figure 5-3. Variation in diametral resilient modulus for FHWA polymer SMA gradation mixes.

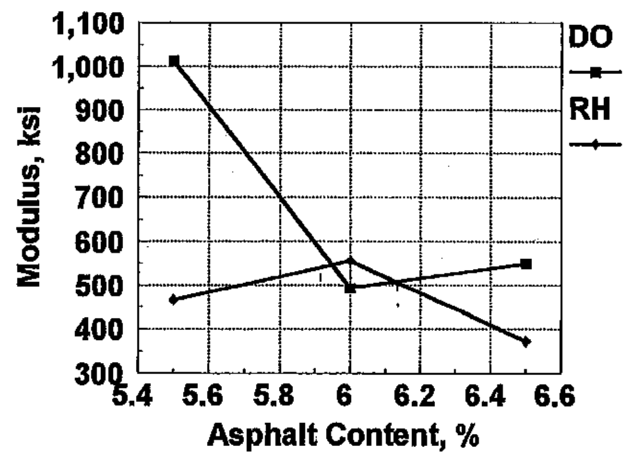

Figure 5-4. Variation in diametral resilient modulus for German SMA gradation mixes.

indicating that the different material elements do not change the general behavioral tendencies of the SMA mixture.

5.2.2 Optimum Asphalt Contents

Table 5-1 presents the average and standard deviations for the resilient modulus testing conducted on the samples prepared at optimum asphalt content. A statistical evaluation of these data show that the crushed gravel mixtures were consistently different from the other mixtures, both SMA and IDOT HMS gradations. There was no statistical difference between the IDOT HMA gradations, and the SMA gradations for similar aggregates. The polymer SMA mixtures demonstrated statistically different modulus values, and were consistently lower than their fiber stabilized counterpart. The German SMA gradations did not produce statistically different modulus values from the other SMA mixtures with fibers.

Table 5-1. Diametral resilient modulus values at optimum asphalt content.

70 F	IDOT HMA	FHWA-SMA	Polymer SMA	German-SMA
Limestone	716,270	557,290	--	--
Dolomite	656,210	590,550	235,646	468,923
Crushed gravel	420,043	334,318	--	--
Rhyolite	569,748	474,590	235,998	504,708
90 F				
Limestone	196,051	155,330	--	--
Dolomite	200,936	182,862	68,498	141,538
Crushed gravel	140,806	80,822	--	--
Rhyolite	131,049	107,074	75,564	135,040

The testing conducted at the two temperature levels provides an indication of the temperature susceptibility of the mixtures as produced by the asphalt binder. It was felt that the fiber stabilized asphalt binders would show a different degree of temperature sensitivity because the Penetration Index, PI, of the asphalt was significantly changed, as shown in

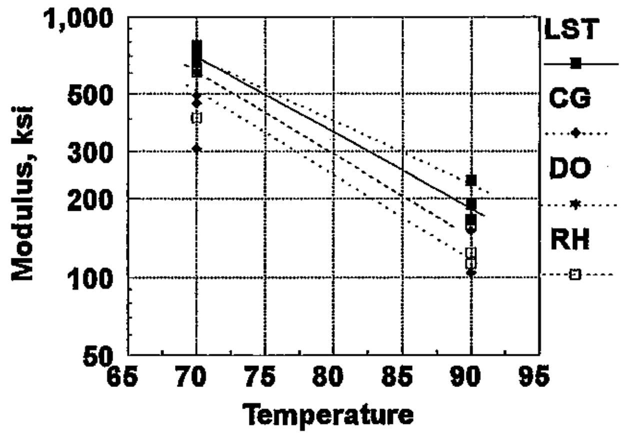

Figure 5-5. Modulus vs temperature for IDOT HMA gradation mixes.

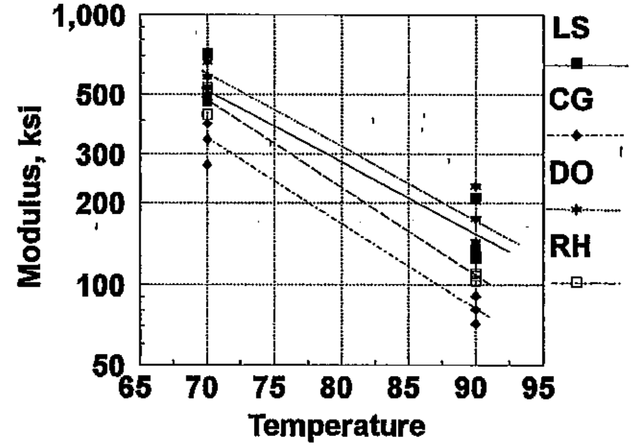

Figure 5-6. Modulus vs temperature for FHWA-SMA gradation mixes.

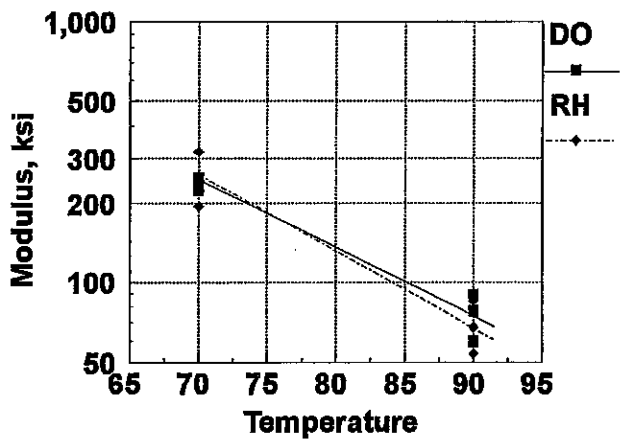

Figure 5-7. Modulus vs temperature for FHWA -SMA polymer mixes

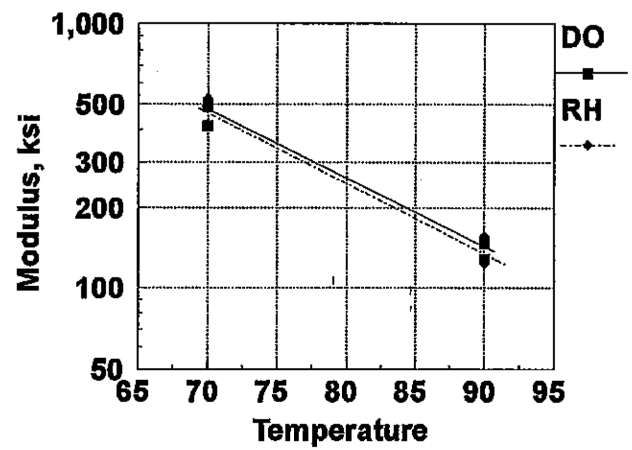

Figure 5-8. Modulus vs temperature for German SMA gradation mixes.

Chapter 4. The plots of resilient modulus vs temperature in Figures 5-5, through Figure 5-8 do not indicate any quantifiable difference in the temperature influence on modulus at normal and higher temperatures. This would provide validation for selecting asphalt characteristics, and stabilization on the basis of asphalt rheology as related to draindown. The rheological differences produced by the different fibers does not significantly alter the high temperature performance.

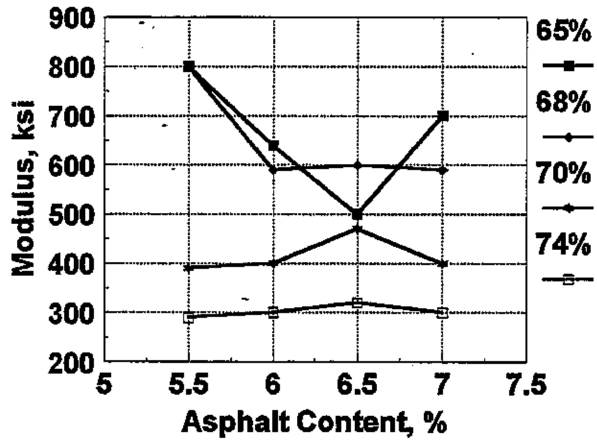

Figure 5-9. Diametral resilient modulus for various sand-stone ratios.

5.2.3 Specialty Studies

Modulus testing on the special mixes that were discussed in Chapter 4 are presented here to provide supportive data for the structural comparisons made on the standard samples.

Figure 5-9 contains the modulus values for the sand/stone combinations using the Limestone coarse aggregate. This testing clearly shows that as the proportion of stone increases, the impact of the asphalt is lessened and the response curve becomes flatter without an apparent hump for use in selecting an optimum value. While the volumetrics were considerably altered by these changes, the resilient modulus values to not show as distinct a change, indicating the predominant influence of the asphalt binder, which is not altered by the change in aggregates.

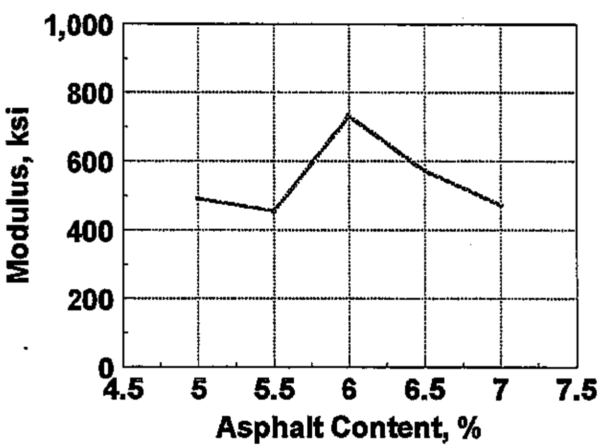

Figure 5-10. Diametral resilient modulus for Fairmount cubical SMA mixture.

Figure 5-10 presents the modulus values for the Limestone aggregate SMA mixtures prepared with all cubical particles, in the 30/70 sand/stone ratio. The resilient modulus values for the cubical particle mixtures is not slightly higher than the standard SMA mixture with the same aggregate. This further supports the contention that volumetrics for an SMA can be successfully used to control the composition of an SMA without sacrificing control over the traditional structural parameter of stiffness.

The compactability samples which indicated the relative attainment of a stable stone skeleton provide an interesting comparison of resilient modulus data. Figure 5-11 presents the resilient modulus data for the IDOT HMA mixtures as a function of compaction effort. Figure 5-12 presents the resilient modulus results for the SMA fiber mixtures, the only samples studied here. The IDOT HMA mixtures indicate a trend for modulus change with increased compaction. It must be recalled that voids were continually reduced with increased compaction effort for these mixtures. Thus, the trend of a decrease in modulus as compaction effort increases is typical of the relationship for a dense graded HMA.

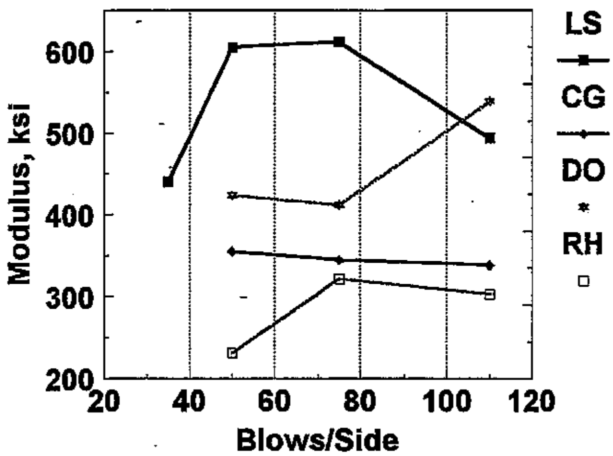

Figure 5-11. Variation in modulus with compactive effort for IDOT HMA gradation mixes.

The SMA mixtures, on the other hand indicate an increase in modulus at 75 blows compaction. This lends support to the observation that 75 blow compaction may provide a more stable skeleton than 50 blow. In all of the SMA mixtures, increased compaction above 75 blows produced no further reduction in air voids, but there was a reduction in modulus for the Limestone aggregate. This could be contributed to the low crushing strength of the limestone aggregate, with a breakdown in aggregate from the increased compaction that is not seen in the other more durable aggregates. The other aggregates do not show as dramatic a decrease in modulus, indicating that the skeleton is relatively constant under

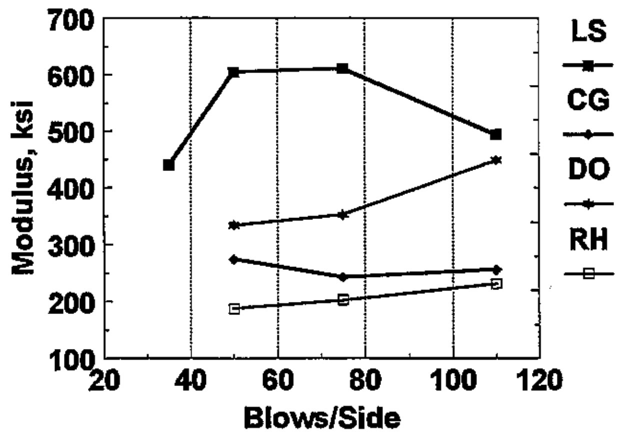

Figure 5-12. Variation in modulus with compactive effort for FHWA - SMA gradation mixes.

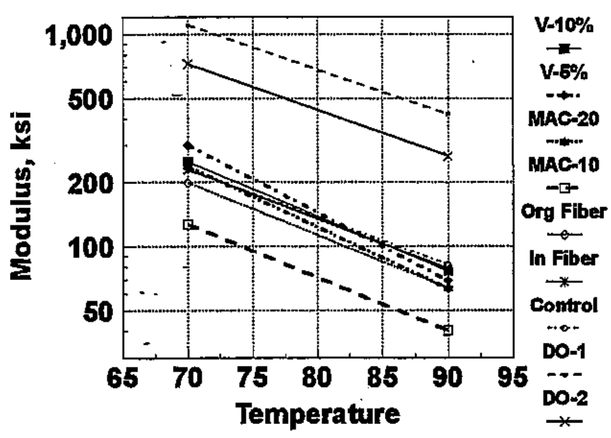

Figure 5-13. Modulus - temperature relationship for field SMA mixtures, Wisconsin and Illinois.

increased compaction efforts for these aggregates. This lends support to the desire to avoid the use of limestone aggregates in an SMA mixture.

The modulus values for the IDOT field mixes were comparable, but slightly higher than the values found in the laboratory, and the temperature relationships are typical. The Wisconsin SMA field mixes were lower than the Illinois field SMA mixes, most likely due to their smaller maximum size aggregate. The modulus temperature relationships for these mixes are shown in Figure 5-13.

5.3 INDIRECT TENSILE STRENGTH

The purpose of this testing was to establish structural adequacy at 70 F for all mixtures. Testing was performed at -20 F to establish any significant difference in low temperature strengths for the different mixes arising through embrittlement of the binder.

5.3.1 Optimum Asphalt Contents

Table 5-2 contains the indirect tensile strength data for the mixtures prepared for this project. The data indicate that suitable strength is being provided by the SMA mixtures to compare favorably with typical IDOT HMA mixtures. Again, the polymer and interstate mixtures have the lowest tensile strength, just as they had the lowest modulus values.

5.3.2 Specialty Mixes

Table 5-3 contains the indirect tensile data for the field mixes tested. The IDOT field SMA mixes were recompacted, while the Wisconsin field SMA mixes were cored from the pavement after less than 3 months of service. These values are typical when compared to the laboratory values. The field mixes had a smaller maximum aggregate size. The binder layers from Wisconsin are the traditional dense graded HMA. Only the surface layers were SMA with the exception of the control.

Table 5-2. Indirect tensile strength, psi.

70 F	IDOT SMA	SMA-F	SMA-P	SMA-Ger
Limestone	206	180	--	--
Dolomite	215	196	125	188
Crushed	149	102	--	--
Rhyolite	168	141	126	152
90 F				
Limestone	153	157	--	--
Dolomite	162	153	95	163
Crushed	128	86	-	--
Rhyolite	138	140	96	122

Table 5-3. Indirect tensile strength, specialty mixes, psi.

Wisconsin Sections	Surface	Binder 1	Binder 2
Vestoplast - 10%	125	162	168
Vestoplast 5%	108	166	150
MAC-20HD	136	178	--
MAC - 10	104	181	160
Organic Fiber	104	143	162
Inorganic Fiber	115	346	159
WisDOT Control - 1	133	162	--
WisDOT Control - 2	149	154	--
IDOT Field SMA	Field Recompacted Surface		
Dolomite-1	176		
Dolomite-2	182		

5.4 PERMANENT DEFORMATION TESTING

The Marshall briquettes were used for repeated load permanent deformation testing. The ends of the samples were ground to a smooth parallel finish. The ends were coated with a light application of vacuum grease to further reduce end effects. These prepared samples were placed in an electro-hydraulic MTS machine and subjected to repeated loadings of either 20 or 40 psi vertical compression. The haversine load pulse consists of 0.1 seconds load application, and 0.9 seconds of rest. The at rest load was set at 1 psi. Sample height measurements were recorded immediately prior to each load pulse and stored on computer for later analysis. Testing was conducted for 5,000 load repetitions, at temperatures of 70, 95, and 120 F.

This test sequence produces a strain-load curve similar to that shown in Figure 5-14. This data can be presented in two formats:

$$\varepsilon_P = A(N)^B \qquad \text{or} \qquad \frac{\varepsilon_P}{N} = A(N)^m$$

where:

A and B are material properties determined from the test data

m equals B-1

ε_p is the permanent strain

N is the number of load repetitions

The strain rate format shown as the second equation here has been selected for this project as it most appropriately lends itself to description of material behavior under repeated loadings. A typical curve is shown in Figure 5-14. This representation provides a unique ability to interpret material behavior when it changes as the number of loadings incresases. Figure 5-15 contains the data plotted in the rutting rate format. This figure contains a stable non-rutting curve, an unstable curve, and a curve that is gaining rut resistance as loadings increase. The slope of these curves taken at each data point, shown in Figure 5-16, clearly indicates the nature of permanent deformation that is developing. A mixture with no rutting whatsoever stabilizes with a

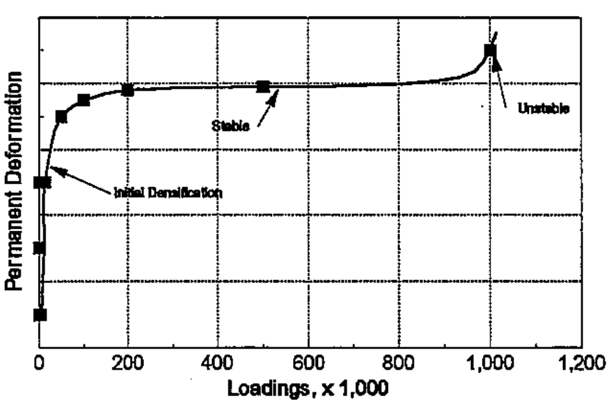

Figure 5-14. Typical permanent deformation curves.

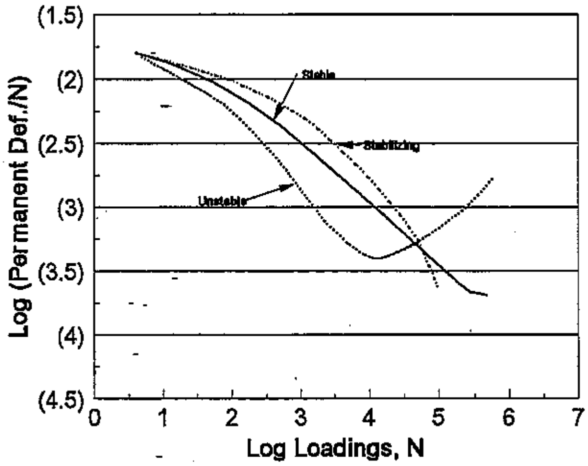

Figure 5-15. Rut rate curves for data in Figure 5-14.

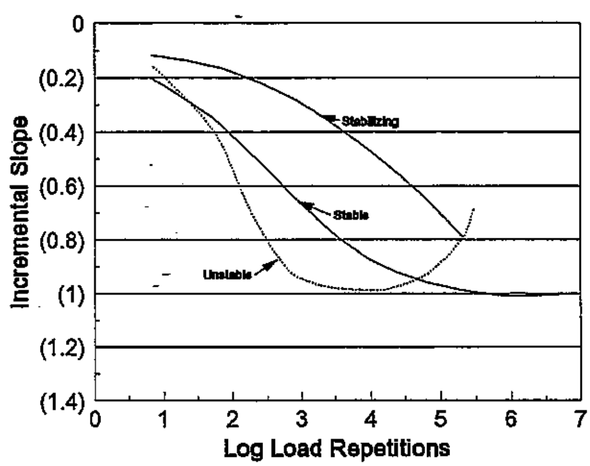

Figure 5-16. Slopes of rutting curves shown in Figure 5-15.

slope of 1.0. Most dense graded HMA mixtures will stabilize with a slope of 0.78, with a lower slope generally indicating more permanent deformation potential. An unstable mixture will gradually develop a slope that decreases and eventually goes positive as the sample eventually fails. This trend in the slope provides a means of evaluating the performance of a mixture in repeated loading before the trend becomes evident in the normal forms of data presentation.

The graphical presentation requires a visual examination of each graph to establish the trend in permanent deformation. The trends exhibited in the data include a stable curve where the mixtures approached a straight line. An unstable curve where the data indicates the beginning of an upward trend in the curve developing late in the test sequence, a stabilizing curve where the data show the mixture to be approaching a straight line relationship, but not quite there, and a filed curve where the mixture became unstable during the early testing sequence.

The A term indicates the initial densification potential for the mixtures, and is most influenced by the initial compaction. This value may differ significantly from lab to field samples, and is not as representative of rutting potential as the slope, m, value.

5.4.1 IDOT HMA Mixtures

The permanent deformation data for the mixtures prepared with the IDOT HMA gradation are shown in Table 5-4. The Fairmount aggregate exhibited the most stable mixture over the temperature range. The dolomite and crushed gravel aggregates exhibited unstable behavior at the high temperature, with the crushed gravel becoming unstable at the intermediate temperature. The rhyolite aggregate exhibited the stabilizing curve, indicative of a mixture exhibiting increased resistance to permanent deformation with increasing loadings, but the mixture failed at the high temperature.

Table 5-4. Permanent deformation results for the IDOT HMA gradations.

		70	95	120
Limestone	m	0.714	0.696	0.675
	Curve	Stable	Stable	Stable
Dolomite	m	0.66	0.694	0.692
	Curve	Stable	Stable	Stable/Unstable
Crushed gravel	m	0.621	0.55	0.639
	Curve	Stable	Failing	Failed
Rhyolite	m	0.505	0.592	0.592
	Curve	Stabilizing	Stabilizing	Failing

5.4.2 SMA Fiber Mixtures

The permanent deformation test results for the FHWA SMA mixtures prepared with fibers are shown in Table 5-5. The trends for these mixtures were similar to that found for the IDOT HMA gradations.

Table 5-5. Permanent deformation results for FHWA-SMA gradations.

		70	95	120
Limestone	m	0.676	0.649	0.662
	Curve	Stable	Stable	Stable
Dolomite	m	0.691	0.673	0.611
	Curve	Stable	Stable/Unstable	Stable/Unstable
Crushed gravel	m	0.451	0.551	0.639
	Curve	Stabilizing	Failing	Failing
Rhyolite	m	--	0.55	--
	Curve	Stabilizing	Failed	Failed

These results indicate a slightly smaller m value, but this value is representative for the entire curve. The SMA permanent deformation curves were highly curvilinear, indicating that more densification was occurring with these mixes during the early loading portion. The slopes for the later loading cycles were considerably higher, and in some instances, approached unity, indicating that these aggregate skeletons were locking up somewhat as the samples densified.

5.4.3 SMA Polymer Mixtures

The permanent deformation results for the Polymer modified binder SMA mixtures is shown in Table 5-6. These mixes demonstrated a small unstable portion at the very end of the loading sequence, not over the entire curve. the results were not dissimilar from the traditional FHWA SMA mixtures, with the Dolomite aggregate producing a less stable mixture than the rhyolite aggregate mixture..

Table 5-6. Permanent deformation results for SMA-Polymer mixes.

Temperature, F		70	95	120
Rhyolite	m	0.641	0.641	0.655
	Curve	Stable	Stable	Un-Stable
Dolomite	m	0.59	0.581	0.611
	Curve	Un-Stable	Un-Stable	Un-Stable

5.4.4 German SMA Mixtures

The results of the permanent deformation testing for the German SMA gradation mixtures is presented in Table 5-7. The results for these mixtures indicate similar stability to the SMA mixtures, with a slightly higher degree of interlock developing under increased loadings, that is not evident from the overall parameters.

5.4.5 Specialty Mixtures

Due to size limitations, not all of the Wisconsin cores could be tested in permanent deformation. The SMA cores were 1.5 inches thick, which did not provide for testing in repeated load configuration with the necessary accuracy. The cores that were tested provided m values ranged from 0.56 to 0.44 for the SMA sections, and the control section was 0.62. The binder mixtures ranged from 0.66 to 0.79, very typical of dense graded HMA.

Table 5-7. Permanent deformation results for the German SMA mixtures.

Temperature, F		70	95	120
Dolomite	m	0.483	0.616	0.617
	Curve	Stable	Un-Stable	Stable
Rhyolite	m	--	0.592	--
	Curve	Stabilizing	Un-Stable	Failed

5.5. RESULTS

The structural testing of the different mixtures indicates that the SMA mixtures provided structural performance related properties that were consistent with the-traditional IDOT high quality dense graded HMA. Modulus and indirect tensile strength values were lower for the SMA mixtures, but still in an acceptable range for a high quality surface mixture. The permanent deformation characteristics were similar for all mixes. The SMA mixtures exhibited some tendencies that indicate they may possess slightly more rutting resistance if the mixtures were designed to utilize the aggregate qualities better, and compacted with an appropriate level of compaction effort to achieve stone on stone skeleton characteristics in the laboratory. The fifty blow compactive effort used was shown to be insufficient for these aggregates to achieve the skeleton required for an SMA mixture.

The permanent deformation testing indicate that the IDOT HMA gradation and aggregates produce a consistent mixture with good resistance to permanent deformation. The influence of aggregate characteristics on structural properties was apparent for the HMA testing in all mixtures. The effect was not as significant for the SMA mixtures where the asphalt binder is a more significant variable, especially in mixtures not compacted to stone on stone interlock, as was demonstrated for the SMA mixtures tested here. The SMA mixtures do appear to be developing more interlock as they densify, as indicated by their more curvilinear shape producing slopes closer to unity at the end of the testing sequence. The dense graded HMA appear to be exhibiting a more uniform rate of rutting with the more linear rut rate curves.

There is no reason to expect different structural properties in a well designed SMA mixture, compared to a dense graded HMA. A factor to investigate in future studies is the difference produced by compaction with the SHRP gyratory compactor to more closely moel field constructed mixtures.

6.0 CONCLUSIONS

6.1 INTRODUCTION

The objectives of this preliminary study of SMA characteristics were to highlight areas of the mixture preparation and characterization procedures currently used and note any deficiencies or areas of concern which should be observed to. Data was developed for four different aggregates, one asphalt cement binder, two additives, and three gradations, as described earlier. Several field mixes were obtained from Illinois and Wisconsin, representing several different mixtures and binders. Data were analyzed and conclusions drawn relative to several areas of the mix design and performance characteristics of SMA mixtures relative to IDOT dense graded HMA.

6.2 DRAINDOWN

The current Schellenberg beaker procedure to evaluate draindown characteristics may not be sufficient for a variety of aggregates. The amount of fiber required to stabilize the binder has been shown in this limited study to be influenced by the aggregate type, with silicate aggregates allowing more flow than carbonate aggregates. The flat plate procedure used in this study appears sensitive enough to differentiate between aggregate types. The new procedure being developed by NCAT may be sufficient. In this study, the binder on the crushed gravel and rhyolite did not stabilize at the lower recommended dosages, which could account for the lower structural properties noted in testing. The crushed gravel mixture was the only mixture which gave statistically different structural properties in all instances. Until further testing confirms this finding, the SMA mixture should not be constructed using siliceous aggregates.

The replacement of the fiber stabilized binder with the polymer modified binder did not improve the performance of the mixtures. The limestone mixture showed instability at the higher temperatures, while the dolomite mixture showed instability at all temperatures in the permanent deformation testing. The structural properties were significantly lower for the polymer mixes.

6.3 PARTICLE SHAPE

The aggregates in this study all met the particle shape requirements for the SMA mixtures. The limestone aggregate contained the largest percentage of flat-elongated particles. This mixture generated good structural properties and resistance to permanent deformation. The removal of all the flat-elongated particles in the limestone aggregate to produce an SMA mixture with all cubical particles produced a mixture with properties very similar to the other SMA mixtures using the 1992 FHWA recommended gradation. Laboratory compaction characteristics of flat elongated limestone particles may produce a very different mixture in the laboratory than actually is obtained in the field due to particle orientation and fracturing characteristics produced during Marshall compaction.

6.4 GRADATION

The FHWA gradation produced acceptable SMA mixtures. The VMA in these mixtures was marginally low for the current recommendations which corresponds to the structural testing performed on mixtures prepared for the limestone aggregate with varying ratios of plus and minus the #4 sieve material. These mixes indicated that increasing the plus #4 material produced a more positive interlock in the aggregate with better voids.

While satisfactory mixes can be obtained following the recommended gradation, there is more investigation required for each aggregate combination used to determine if the recommended gradation can be improved by altering the proportion of stone and matrix sized particles. Recommended changes to the FHWA SMA gradation proposed in 1993-94 would increase the percent of stone in the gradations.

6.5 AGGREGATE TYPE

The four different aggregates produced similar mixtures with the exception of the crushed gravel which consistently produced lower structural behavior. This is felt to be due principally to the aggregate surface - binder interaction and the demonstrated effect of the fiber stabilization. This reduced property tendency was evident in all mixes with the crushed gravel aggregate. The limestone aggregate produced the most consistent SMA mixture, but would probably not be recommended because of the aggregate durability considerations. The benefits of this aggregate most likely stem from the angularity characteristics which were significantly different from the others which were similar to each other. The crushed gravel aggregate did not perform as well as the other aggregates in this study, indicating further study should be performed before this aggregate type is recommended for wide usage in SMA mixtures.

6.6 COMPACTION AND VOID LEVELS

The compaction study clearly demonstrated that laboratory compaction using fifty blow Marshall compaction is not sufficient to lock the stone on stone skeleton for these aggregates blended to the center of the respective gradation bands. Seventy-five blow Marshall compaction produced a skeleton that was resistant to further densification at higher compactive efforts. This laboratory effect could be expected to produce samples with different structural characteristics than field cores. Seventy five blow Marshall compaction should produce more similar samples. Gyratory compaction is not felt to be necessary because the compaction characteristics of an SMA are not at all similar to the dense graded HMA mixtures which require extensive shearing compaction and extensive particle reorientation before final density is achieved. SMA mixtures only receive compaction to the extent that the larger stone interlock is achieved. The matrix is not worked to achieve densification, and indeed, in an SMA mixture it cannot be worked extensive during compaction because of the stone interlock.

The standard IDOT dense graded HMA gradations produced typical compactability curves indicating that densification would continue at higher blow counts with no apparent leveling of the density increase being indicated. The void levels currently recommended appear sufficient for SMA mix design. Close examination of the voids in the compacted samples at different asphalt contents can provide a preliminary indication of the functioning attained in the stone on stone skeleton. The SMA gradation should produce a mixture with a much smaller change in VMA as asphalt content changes than is normally exhibited in a normal dense graded HMA. If VMA changes appear similar to those seen in HMA mixes, the stone on stone skeleton is not developing, and the sand matrix is being compacted, and not filling the voids in the stone skeleton.

The mix design procedure should be modified to evaluate the attainment of stone on stone interlock for each mixture. This could indicate the need for a seventy-five blow design rather than a fifty blow mix design to more adequately lock the stone particles together in a manner simulating field construction.

6.7 STRUCTURAL CHARACTERISTICS

Most traditional structural characterization tests accentuate the tensile properties of the mixtures. For this reason, they are typically lower for the SMA mixtures because of the significantly higher binder content. The diametral resilient modulus and indirect tensile strength are lower, but the small decrease does not indicate decreased performance as the SMA mixtures are proposed only for use in the surface layer, not the binder layers where tensile characteristics become important.

The stabilized binder in the SMA mixtures did not provide a different degree of temperature susceptibility. The change in modulus or tensile strength with temperature was similar for SMA and IDOT HMA mixtures. The ductility at the lower temperatures was not measured during the indirect tensile test at 20 F. This could be increased in the polymer modified binders which would increase resistance to low temperature cracking. This was not substantiated in this study.

The permanent deformation testing showed similar characteristics between the SMA and IDOT HMA mixtures. This is true even though the SMA mixtures evidently were not compacted to stone on stone interlock with the fifty blow Marshall compaction used. The SMA mixtures appeared to gain stability under repeated loadings while the IDOT HMA gradations showed typical dense graded HMA performance with the gradual steady accumulation of permanent deformation under increased loadings. Compaction in the laboratory that completelyl developed the stone on stone contact would demonstrate this superior resistance to permanent deformation over the entire loading history.

A properly constructed SMA could be expected to provide rutting resistance better than the IDOT HMA mixture. The key is to achieve the stone on stone skeleton during construction. This requires that field compaction of the SMA be completed to a higher level than traditionally used for the dense graded HMA mixtures, that is, require 96 to 97 percent compaction be achieved in the field.

All mixtures appeared to lose stability at the higher temperatures with about the same degree of consistency. Some samples remained stable, while some became unstable, from the same mix. Because the repeated load test is a compressive test, the binder effects are minimized and the aggregate structure is emphasized in the test. Thus, no binder effects could be seen in the testing. The aggregate structure for the SMA and IDOT HMA mixtures provide similar rutting resistance. The German SMA gradation appears to provide a slight increase in rutting resistance. This is most likely due to the increased stone content in this gradation, a feature shown to be desirable in the other SMA gradations.

6.8 FIELD SMA MIXES

The field SMA mixes from Wisconsin and Illinois produce similar structural characteristics when allowances are made for aggregate size differences. The permanent deformation testing produced similar trends indicating that the SMA mixtures require some number of loads to develop a full stone on stone skeleton to resist rutting. This may relate to laboratory mix-design compaction levels as mentioned earlier, although no information could be produced to support this idea. The IDOT field mixes showed excellent compaction characteristics, with minor compaction at higher compactive efforts, and should perform adequately in the field with little further densification.

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