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Superpave Asphalt Mixture Design

Workshop
Workbook

Superpave

Asphalt Mixture Design

Workshop

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With excerpts from

FHWA National Asphalt Training Center Training Manuals,
Asphalt Institute MS & SP Series, &

Superpave Lead States Guidelines.

This workshop is intended to demonstrate the Superpave asphalt mixture design system developed by the Strategic Highway Research Program *(SHRP)*. This workshop includes the latest recommendations of the Superpave Lead States and the Mixture/Aggregate & Binder Expert Task Groups.

Very special thanks to:

Dr. Aroon Shenoy, Ph.D.
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Superpave Asphalt Mixture Design

FOREWORD

The focus of this workbook is to provide engineers and technicians with a detailed example of Superpave *Volumetric* asphalt mixture design.

INTRODUCTION

a. Background of SHRP

The Strategic Highway Research Program *(SHRP)* was established by Congress in 1987 as a five-year, $150 million dollars, product driven, research program to improve the quality, efficiency, performance, and productivity of our nation's highways and to make them safer for motorists and highway workers. It was developed in partnership with States, American Association of State Highway and Transportation Officials *(AASHTO)*, Transportation Research Board *(TRB)*, Industry, and Federal Highway Administration *(FHWA)*. SHRP research focused on asphalt (liquids and mixtures), concrete & structures, highway operations, and long-term pavement performance *(LTPP)*.

b. SHRP Implementation

As a follow-up program to SHRP, Congress authorized $108 million over six years as part of the Intermodal Surface Transportation Efficiency Act *(ISTEA)* of 1991, to establish programs to implement SHRP products and to continue SHRP's LTPP program. The FHWA was given the responsibility of directing the implementation efforts to facilitate the application of the research findings. Several concurrent efforts were undertaken including:

(1)	TRB Superpave Committee	
(2)	TRB Expert Task Groups:	
	(a) Asphalt Binder	
	(b) Asphalt Mixture/Aggregate	
	(c) Communications	
	(d) Superpave Models	- NCHRP 9-19
(3)	Pooled Fund Equipment Buys	- Nearly Completed
(4)	National Asphalt Training Center	- Completed
(5)	Mobile Superpave Laboratory	
(6)	Equipment Loan Program	- Completed
(7)	Expert Technical Assistance	
(8)	Superpave Regional Centers	
(9)	Superpave Models Contract	- NCHRP 9-19
(10)	Superpave Lead States	- Twilighted Sept. 00

In 1998, Congress enacted the Transportation Equity Act for the 21st Century *(TEA21)*. Although TEA21 encourages the continued implementation of SHRP technologies, no specific funding is provided. To address this shortfall in funding the FHWA, AASHTO, TRB, and NCHRP approached the States to fund critical Superpave activities with NCHRP funding. The Asphalt TWG has been replace by the TRB Superpave Committee. The ETG's have also been transferred to TRB for management. FHWA will continue to provide expert technical assistance.

SUPERPAVE OVERVIEW

The final product of the SHRP asphalt program area is Superpave. Superpave is an acronym which stands for:

Superior **Per**forming Asphalt **Pave**ments.

Superpave is a performance-related asphalt binder and mixture specification. Superpave is not just a computer software package, nor just a binder specification, nor just a mixture design and analysis tool. Superpave is a **system** which is inclusive of all these parts.

Superpave mixture design provides for a functional selection, blending, and volumetric analysis of proposed materials, along with an evaluation of moisture sensitivity. There are **four steps** in mixture design:

① Selection of Materials,
② Selection of a Design Aggregate Structure,
③ Selection of the Design Asphalt Binder Content, and
④ Evaluation of Moisture Sensitivity of the Design Mixture.

Criteria for materials selection and compaction are a function of three factors:

a. Environment,
b. Traffic, and
c. Pavement Structure.

Binder selection is based on environmental data, traffic level and traffic speed. Aggregate selection is based upon layer location, traffic level, and traffic speed.

Selection of the design aggregate structure *(design blend)* consists of determining the aggregate stockpile proportions and corresponding combined gradations of the mix design. The design aggregate structure, when blended at the optimum asphalt binder content, should yield acceptable volumetric properties based on the established criteria.

Selection of the design *(optimum)* asphalt binder content consists of varying the amount of asphalt binder in the design aggregate structure to obtain acceptable volumetric properties when compared to the established mixture criteria. It also provides a feel for the sensitivity of the design properties to changes in the asphalt binder content during production.

Evaluation of moisture sensitivity consists of testing the design mixture by AASHTO T-283, or other State specified method, to determine if the mixture will be susceptible to moisture damage.

Simulation Background

a. This simulated project is located in the city of **Hot Mix, USA**.

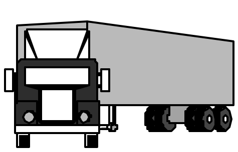
b. The estimated, **20-year**, design traffic for this project is 6,300,000 $ESAL_{80\text{-}kN}$, (18-kip ESAL = 80-kN ESAL).

c. The posted traffic speed for the design section is 80 kilometers per hour, kph (50 mph). The estimated actual average speed for this section, accounting for speeding and rush hour, is 72 kph (45 mph).

d. The mix is a surface course *(such that the top of this pavement layer from the surface is less than 100 millimeters)*.

The project location in conjunction with the Weather Database will provide the minimum pavement temperature, the maximum pavement temperature, and the maximum air temperature. The estimated traffic and project temperature data, in combination with the layer location will establish the material and compaction criteria.

Update: All Superpave mixes are designed volumetrically. Currently under NCHRP study 9-19, *"Superpave Models Development,"* being conducted by the University of Maryland and the University of Arizona, a simple performance test is being identified/developed. The simple performance test will be used in conjunction with the Superpave volumetric mixture design. This test is intended to add an additional level of reliability to assure design mixes are able to resist the applied trafficking with minimal permanent deformation *(rutting)*.



SELECTION OF MATERIALS

The performance grade *(PG)* binder required for the project is based on environmental data, traffic level and traffic speed. The environmental data is obtained by converting historic air temperatures to pavement temperatures. The SHRP researchers developed algorithms to convert high and low air temperatures to pavement temperature. These algorithms have been refined and updated by LTPP:

Refinement

The original SHRP low-pavement-temperature algorithm did not correctly determine the low pavement temperature from the air temperature. The FHWA LTPP program developed a new low-pavement-temperature algorithm from their weather stations at over 30 sites all over North America. Data supporting the LTPP algorithm is presented in *LTPP Seasonal Asphalt Concrete Pavement Temperature Models*, FHWA-RD-97-103, September, 1998.

LTPP High-Temperature Model with Reliability

$$T(pav) = 54.32+0.78\ T(air) -0.0025\ Lat^2 -15.14 \log_{10}(H + 25)+z\ (9 +0.61\ \sigma_{air}^2)^{1/2}$$

where:
- $T(pav)$ = High pavement temperature below the surface, ˚C
- $T(air)$ = High air temperature, ˚C
- Lat = Latitude of the section, degrees
- H = Depth from surface, mm
- σ_{air} = Standard deviation of the high 7-day mean air temperature, ˚C
- z = From the standard normal distribution table, z=2.055 for 98% reliability

LTPP Low-Temperature Model with Reliability

$$T(pav) = -1.56+0.72\ T(air) -0.004\ Lat^2 +6.26\ \log_{10}(H + 25)-z\ (4.4 +0.52\ \sigma_{air}^2)^{1/2}$$

where:
- T(pav) = Low pavement temperature below the surface, ˚C
- T(air) = Low air temperature, ˚C
- Lat = Latitude of the section, degrees
- H = Depth from surface, mm
- σ_{air} = Standard deviation of the high 7-day mean air temperature, ˚C
- z = From the standard normal distribution table, z=2.055 for 98% reliability

A complete report documenting the research is available entitled, *"LTPP Seasonal Asphalt Concrete (AC) Pavement Temperature Models."* Publication No. FHWA-RD-97-103, September 1998.

The average 7-day maximum pavement temperature (T_{max}) and the minimum pavement temperature (T_{min}) define the binder laboratory test temperatures. A factor of safety can be incorporated into the performance grading system based on temperature reliability. The 50 % reliability temperatures represent the straight average of the weather data. The 98 % reliability temperatures are determined based on the standard deviations of the low ($\sigma_{Low\ Temp}$) and high ($\sigma_{High\ Temp}$) temperature data. From statistics, 98 % reliability is two standard deviations from the average value, such that:

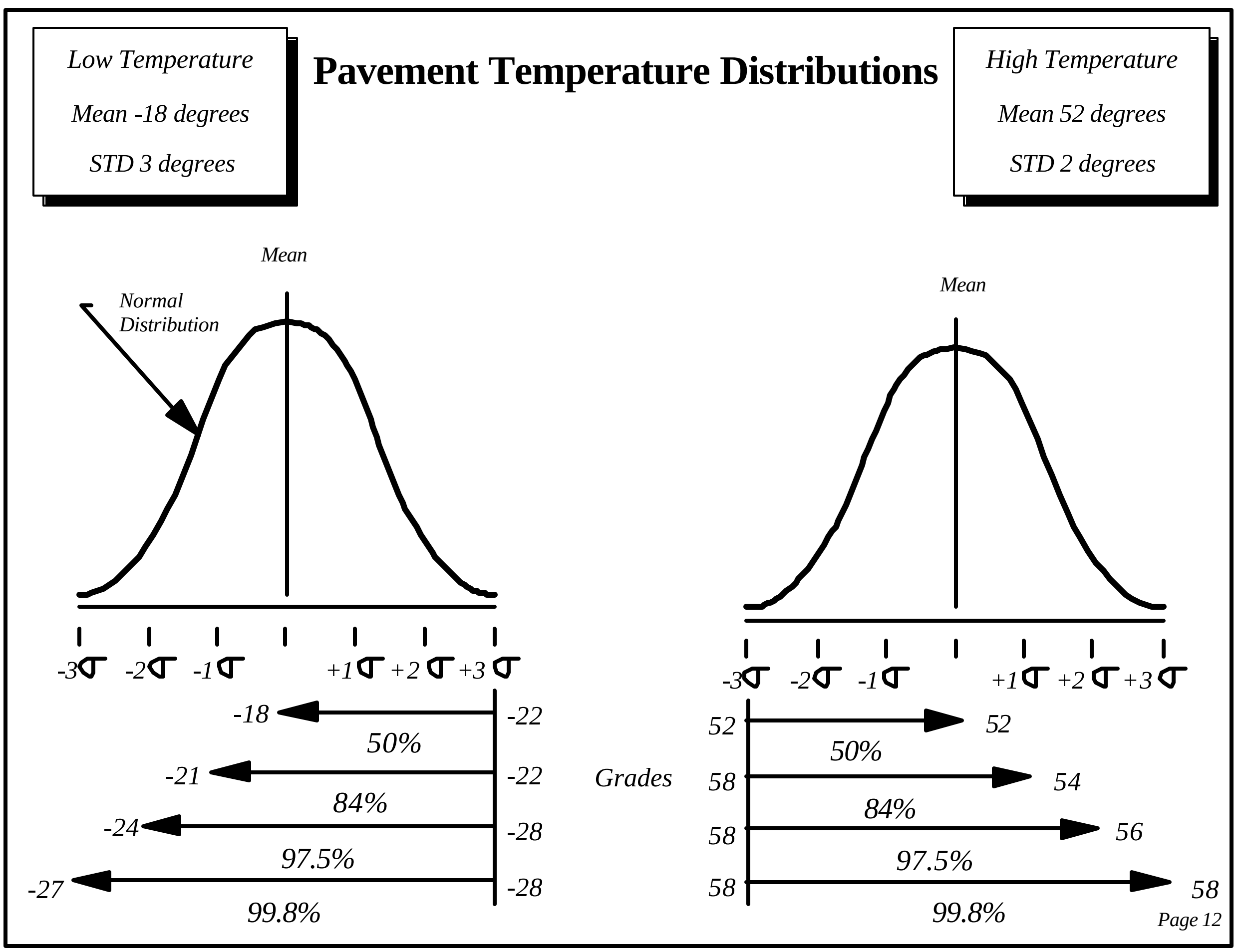
$$T_{max\ at\ 98\%} = T_{max\ at\ 50\%} + 2 * \sigma_{High\ Temp}$$

$$T_{min\ at\ 98\%} = T_{min\ at\ 50\%} - 2 * \sigma_{Low\ Temp}$$

Traffic level and speed are also considered in selecting the project performance grade (PG) binder either through reliability or "grade bumping." A table is provided in AASHTO MP-2, *"Standard Specification for Superpave Volumetric Mix Design,"* to provide the designer with guidance on grade selection.

MP-2, Table 1 - Binder Selection on the Basis of Traffic Speed and Traffic Level

Design ESALs[1] (million)	Adjustment to Binder PG Grade[5]		
	Traffic Load Rate		
	Standing[2]	Slow[3]	Standard[4]
< 0.3	-[6]	-	-
0.3 to < 3	2	1	-
3 to < 10	2	1	-



10 to < 30	2	1	-[6]
≥ 30	2	1	1

(1) Design ESALs are the anticipated project traffic level expected on the design lane over a 20 year period. Regardless of the actual design life of the roadway determine the design ESALs for 20 years and choose the appropriate N_{design} level.
(2) Standing Traffic - where the average traffic speed is less than 20 km/h.
(3) Slow Traffic - where the average traffic speed ranges from 20 to 70 km/h.
(4) Standard Traffic - where the average traffic speed is greater than 70 km/h.
(5) Increase the high temperature grade by the number of grade equivalents indicated (1 grade equivalent is 6°C). Use the low temperature grade as determined in Section 5.
(6) Consideration should be given to increasing the high temperature grade by 1 grade equivalent.

Note 4 - Practically, performance graded binders stiffer than PG 82-XX should be avoided. In cases where the required adjustment to the high temperature binder grade would result in a grade higher than a PG 82, consideration should be given to specifying a PG 82-XX and increasing the design ESALs by one level (e.g., 10 to < 30 million increased to ≥ 30 million).

Author's Note

The designer should use either reliability or the above table to address high traffic levels and slower traffic speeds. Both methods can effectively "bump" the performance grade such that the appropriate binder is used. However, using them in combination will result in an unnecessarily stiff binder, which in turn may cause problems during production and lay down.

Performance grades are delineated by 6˚C increments. The following table shows the Superpave performance grade temperatures. A few State highway agencies have chosen to specify alternative performance grades. In Georgia, for example, the department of transportation specifies a PG 67-22. This ensures the DOT of receiving an asphalt binder similar to what they have used historically, AC-30. Although highway agencies are not encouraged to alter the Superpave performance grades, Georgia is still receiving a performance grade asphalt. Binders provided to meet their modified specification still have to meet the Superpave test criteria, just at different temperatures.

Table: Superpave Performance Grades (PG)

Average 7-day Maximum Pavement Temperature (PG ##-__)

46°C	52°C	58°C	64°C	70°C	76°C	76°+ n6°
Minimum Pavement Temperature (PG __-##)						
+2°C	-4°C	-10°C	-16°C	-22°C	-28°C	-28°-n6°

For Hot Mix, USA, the following data is obtained from the project location and historical temperature data:

a. Latitude is 41.1 degrees,
b. 7-day average maximum air temperature is 33.0˚C with a σ of 2˚C, and
c. 1-day average minimum air temperature is -21.0˚C with a σ of 3˚C.

From this data the high and low pavement temperature are determined at a depth of 20 mm:

High pavement temperature **50.8˚C**
Low pavement temperature **-14.7˚C**

PG 52-16 at 50% reliability
PG 58-22 at 98% reliability

Q. Does the project traffic level of 6.3 million ESAL's warrant an increase in the high temperature performance grade?

a. Yes, or
b. No.

Q. Does the estimated average speed of 72 kph warrant an increase in the high temperature performance grade?

a. Yes, or
b. No.

For Hot Mix, USA, the 50 % reliability performance grade is a PG 52-16. The project traffic level and speed do not require grade bumping. However, the traffic speed is just above the threshold for grade bumping and historically in this area pavements have shown susceptibility to low-temperature cracking. Such that, the agency shall require a **PG 58-22**.

Binder Selection

The project asphalt binder is tested for specification compliance to the Superpave PG system.

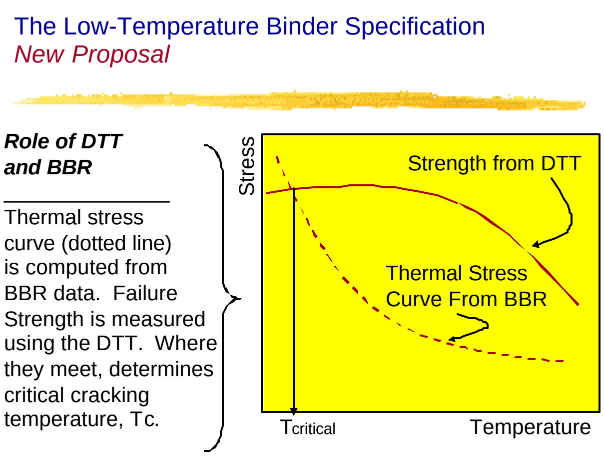
Project Binder: *PG 58-22*
Binder Source: *Asphalt is Us*

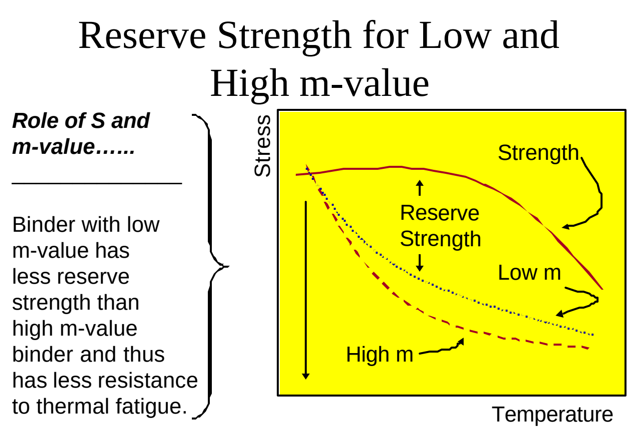
Table: Binder Specification Test Results

Test	Property	Results	Criteria
Original Binder			
Flash Point	n/a	310 ˚C	≥ 230°C
Rotational Viscometer	135°C	0.364 Pa-s	≤ 3 Pa-s
Rotational Viscometer	165°C	0.100 Pa-s	n/a
Dynamic Shear Rheometer, $G^*/\sin\delta$	**58°C**	1.7 kPa	≥ 1.0 kPa
RTFO Residue - Aged Binder			
Mass Loss	n/a	0.4 %	≤ 1.0 %
Dynamic Shear Rheometer, $G^*/\sin\delta$	**58°C**	2.8 kPa	≥ 2.2 kPa
RTFO + PAV Residue - Aged Binder			
Dynamic Shear Rheometer, $G^* \sin\delta$	**22°C**	3.4 MPa	≤ 5 MPa
Bending Beam Rheometer, Stiffness	**-12°C**	280 MPa	≤ 300 MPa
Bending Beam Rheometer, m-value	**-12°C**	0.334	≥ 0.300

Binder ETG - AASHTO MP1(a) - 2001

The Superpave low temperature binder specification has been revised using a new scheme to determine the critical thermal cracking temperature . The main consideration in the new scheme is to unite the rheological properties obtained using the Bending Beam Rheometer (BBR) and the failure properties acquired from the new Direct Tension Test (DTT). The low-temperature task group (LTTG), under the auspices of the Binder Expert Task Group, evaluated the following scheme to define the new low-temperature criteria. The schematic in figure below shows the impact of S(60), m(60), and the failure strength on the thermal cracking behavior of asphalt binders. The thermal stress curve in the figure can be approximated using the BBR data, whereas the failure strength is obtained from the DTT. The critical temperature is determined, as shown, from the thermal stress curve and the strength. The LTTG validated the new scheme using performance data from the Canadian Lamont sections.





Binder Tests Required for Mixture Design

The rotational viscometer (Brookfield™), as part of the binder specification, is performed on the original/unconditioned binder at 135°C. The specification recommends all binders to have a viscosity less than 3 Pascal-seconds (Pa-s). This is to ensure pump-ability during production. For mixture design, the rotational viscometer must be run at a second temperature, typically 160°C. This is done in order to determine the proper mixing and compaction temperatures. SHRP adopted the Asphalt Institute mixing and compaction guidelines base on the temperature-viscosity relationship of the binder, where:

Range for mixing =150 to 190 centiStokes

Range for compaction =250 to 310 centiStokes

The rotational viscometer measures viscosity in centipoises (cP) and the values are reported in Pascal-seconds (Pa-s). The conversion from centipoises to Pascal-seconds is as follows:

1 Pa-s = 1000 centipoises

The relationship between centiStokes and Pascal-seconds (or centipoises) is a function of the asphalt binder specific gravity. The specific gravity of an asphalt binder is a function of temperature. The asphalt binder specific gravity (G_b) is determined according to AASHTO T 228 and is typically measured at 25°C. Tables of G_b temperature correction factors have been developed to adjust G_b over a range of temperatures. The following equation has been determined from these tables:

Correction Factor, CF = -0.0006 (T_{test}) + 1.0135

where: CF is the correction factor, and
T_{test} is the test temperature in °C.

Such that the conversion from centipoises or Pascal-seconds to centiStokes is performed as follows:

$$\frac{X\ cP}{(CF + G_b)} = Y\ cSt \quad or \quad X\ Pa\text{-}s \ast \frac{1000}{(CF + G_b)} = Y\ cSt$$

The project PG 58-22 provides the following test results:

G_b = 1.030

Viscosity at 135°C = 364 cP = 0.364 Pa-s
Viscosity at 160°C = 100 cP = 0.100 Pa-s

Q. What are the equal-viscous mixing and compaction ranges for this asphalt binder?

A. First the temperature correction factors for G_b are calculated at the two test temperatures:

$CF_{135°C}$ = -.0006(135°C) + 1.0135 = 0.933

$CF_{160°C}$ = -.0006(160°C) + 1.0135 = 0.918

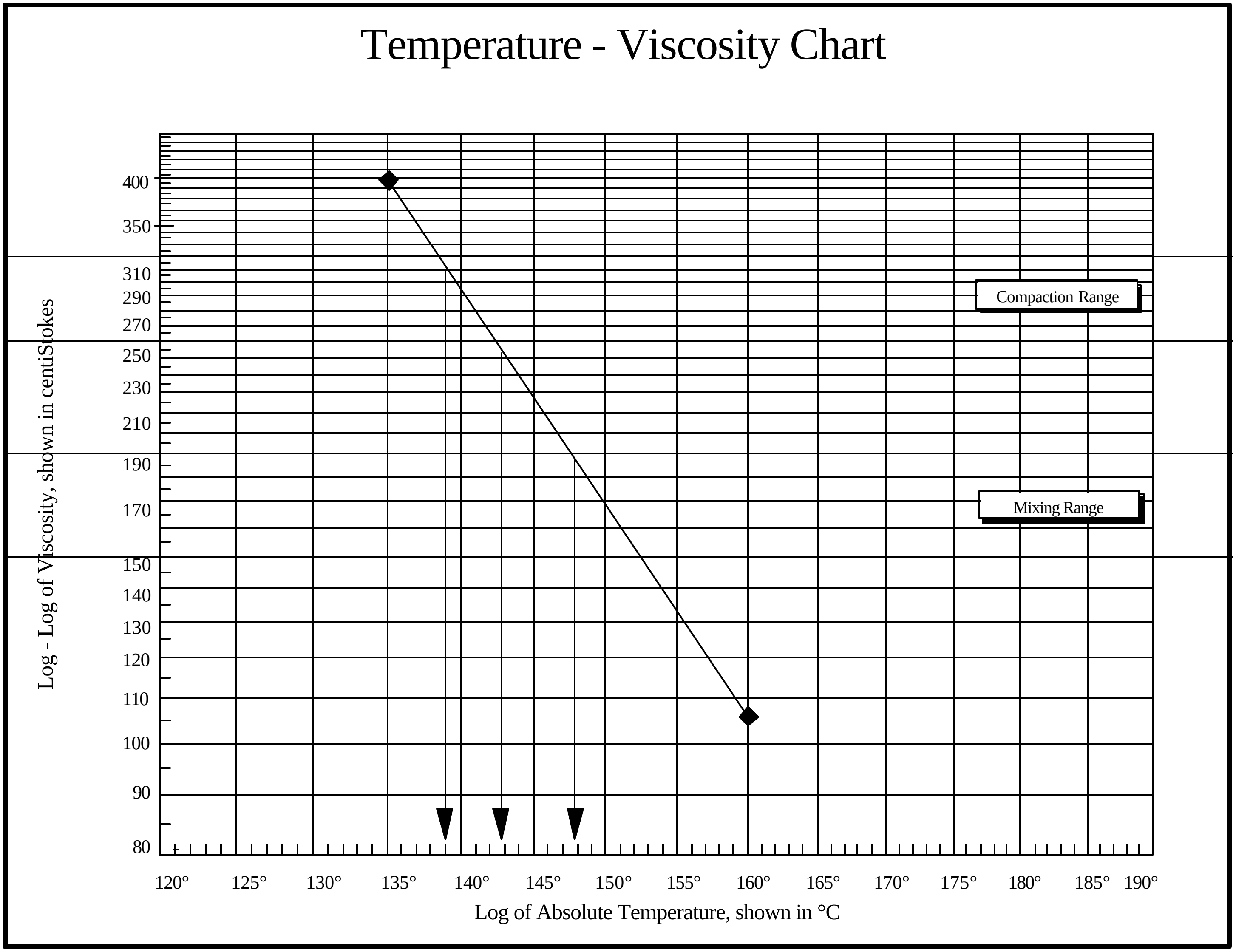
The test results are then converted from Pascal-seconds to centiStokes:

$$\textit{Viscosity at } 135°C = \frac{364\ cP}{(0.933 * 1.030)} = 379 \textit{ centiStokes}$$

$$\textit{Viscosity at } 160°C = \frac{100\ cP}{(0.918 * 1.030)} = 106 \textit{ centiStokes}$$

This data is now analyzed graphically based on the Log-Log$_{(\text{base } 10)}$ of the viscosity in centiStokes plotted against the Log$_{(\text{base } 10)}$ of the temperature in degrees Kelvin (273° + °C), see figure. From the graph the following temperature data is determined:

Range	Temperature, °C
Mixing	148°C to ____?
Compaction	138°C to 142°C


Temperature - Viscosity Chart
Log - Log of Viscosity, shown in centiStokes
400
350
310
290
270
250
230
210
190
170
150
140
130
120
110
100
90
80
Compaction Range
Mixing Range
120°
125°
130°
135°
140°
145°
150°
155°
160°
165°
170°
175°
180°
185°
190°
Log of Absolute Temperature, shown in °C

Summary of Results	
Mixing Temperature Range	148°C to 152°C
Compaction Temperature Range	138°C to 142°C

Note: This relationship ***does not*** *work for all modified asphalt binders.*

Note: See the Appendix for the mathematics required to perform the mixing and compaction temperature range determinations.

Note: The conversion from centipoise to centiStokes is important, however it is not required. Determining mixing and compaction temperatures based upon 150 to 190 centipoise and 250 to 310 centipoise ranges, respectively, will only effect the results by 1 to 2°C.

Aggregate Selection

> *Superpave utilizes a completely new system for testing, specifying, and selecting asphalt binders. While no new aggregate tests were developed, current methods of selecting and specifying aggregates were refined and incorporated into the Superpave design system. Superpave asphalt mixture requirements were established from currently used criteria.*

For this simulated project, four (4) stockpiles of materials consisting of two (2) coarse materials and two (2) fine materials are employed. Representative samples of the materials are obtained, and washed sieve analysis is performed for each aggregate. The gradation results are shown in the Aggregate Blending Section.

The specific gravities (bulk G_{sb} and apparent G_{sa}) are determined for each aggregate. The specific gravities are used in trial binder content and Voids in Mineral Aggregate (VMA) calculations.

Table: Aggregate Stockpiles

Aggregate Stockpile	Bulk, G_{sb}	Apparent, G_{sa}
Coarse Aggregate	2.567	2.680
Intermediate Aggregate	2.587	2.724
Manufactured Fines	2.501	2.650
Natural Fines	2.598	2.673

In addition to sieve analysis and specific gravity determinations, Superpave requires certain consensus and source aggregate tests be performed to assure that the combined aggregates selected for the mix design are acceptable. The consensus property criteria are fixed in the Superpave design system; these are minimum requirements which should be adhered to regardless of geographic location. The source property criteria are specified by the State highway agency. Superpave recommends three source property tests which should be included in the aggregate selection process.

Table: Aggregate Tests

Consensus Properties	**Source Properties (Set by SHA)**
° Coarse Aggregate Angularity (ASTM D 5821) ° Uncompacted Void Content of Fine Aggregate (AASHTO TP 33) ° Flat & Elongated Particles (D 4791) ° Sand Equivalent (T 176)	° Resistance to Abrasion (T 96) ° Soundness (T 104) ° Clay Lumps & Friable Particles (T 112)

Superpave requires the consensus and source properties be determined for the design aggregate blend. The aggregate criteria are based on combined aggregates rather than individual aggregate components. However, it is recommended the tests be performed on the individual aggregates until historical results are accumulated and also to allow for the blending of the aggregates in the mix design.

Author's Note

An aggregate which does not individually comply with the criteria is not eliminated from the aggregate blend. However, its percentage of use in the total aggregate blend is limited.

CONSENSUS PROPERTY STANDARDS

Coarse Aggregate Angularity (ASTM D 5821)

This property ensures a high degree of aggregate internal friction and aids in rutting resistance. It is defined as the percent by weight of aggregates larger than 4.75 millimeters with one or more fractured faces, ASTM D 5821, *"Determining the Percentage of Fractured Particles in Coarse Aggregate."* Where:

"Fractured Face, an angular, rough, or broken surface of an aggregate particle created by crushing, by other artificial means, or by nature (ASTM D 8). A face will be considered a 'fractured face' only if it has a projected area at least as large as one quarter of the maximum projected area (maximum cross-sectional area) of the particle ***and the face has sharp and well defined edges****; this excludes small nicks."*

Fine Aggregate Angularity as Determined by: Uncompacted Void Content of Fine Aggregate (AASHTO TP33) - (Method A)

Uncompacted void content is related to particle shape, angularity, and surface texture. These properties ensure a high degree of fine aggregate internal friction and aid in rutting resistance. Uncompacted void content is defined as the percent air voids present in loosely compacted aggregates smaller than 2.36 mm. Higher void contents correspond to higher fractured faces. A test procedure currently promulgated by the National Aggregates Association is used to measure this property. In the test, a sample of fine aggregate is poured into a small calibrated cylinder by flowing through a standard funnel. By determining the weight of fine aggregate (W) in the filled cylinder of known volume (V), void content can be calculated as the difference between the cylinder volume and fine aggregate volume collected in the cylinder. The fine aggregate bulk specific gravity (G_{sb}) is used to compute fine aggregate volume:

$$\textit{Uncompacted Voids, } U = \frac{(V - \frac{W}{G_{sb}})}{V} * 100$$

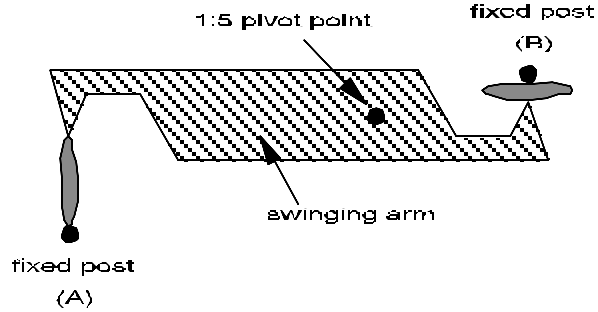
Flat/Elongated Particles as determined by: Flat or Elongated Particles in Coarse Aggregate (ASTM D 4791)

This characteristic is the percentage by weight of coarse aggregates that have a maximum to minimum dimension-ratio greater than five. Elongated particles are undesirable because they have a tendency to break during construction and under traffic. The test procedure, ASTM D 4791, "Flat or Elongated Particles in Coarse Aggregate," is performed on coarse aggregate larger than 9.5 millimeters.

The procedure uses a proportional caliper device (see figure below) to measure the dimensional ratio of a representative sample of aggregate particles. In the figure, the aggregate particle is first placed with its largest dimension between the swinging arm and fixed post at position A. The swinging arm then remains stationary while the aggregate is placed between the swinging arm and fixed post at position B. If the aggregate fits within this gap, then it is counted as a flat/elongated particle.

Note: Superpave uses a <u>single</u> measurement be made for flat/elongated particles. The 5:1 ratio refers simply to the maximum to minimum dimension.

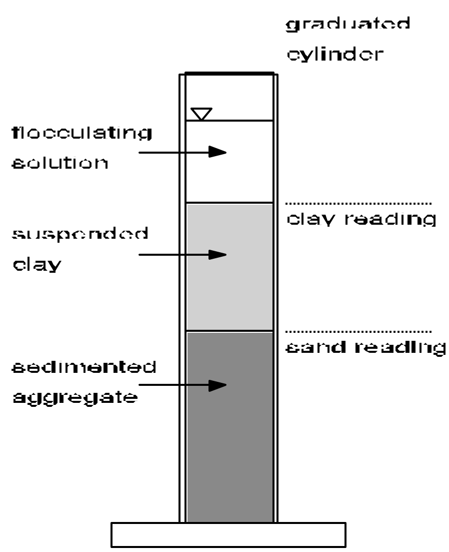
Figure: ASTM D 4791



Clay Content as determined by:
Sand Equivalent Test (AASHTO T 176)

Clay content is the percentage of clay material contained in the aggregate fraction that is finer than a 4.75 mm sieve. It is measured by AASHTO T 176, "Plastic Fines in Graded Aggregates and Soils by Use of the Sand Equivalent Test." In this test, a sample of fine aggregate is placed in a graduated cylinder with a flocculating solution and agitated to loosen clay fines present in and coating the aggregate. The flocculating solution forces the clay material into suspension above the granular aggregate. After a period that allows sedimentation, the cylinder height of suspended clay and sedimented sand is measured (figure below). The sand equivalent value is computed as a ratio of the sand to clay height readings expressed as a percentage.

Figure: AASHTO T 176



Clay Content (Sand Equivalent, SE), -4.75 mm

$$SE = \frac{SR}{CR} * 100$$

SR - sand reading
CR - clay reading

MP-2, Table 4 - Superpave Aggregate Consensus Property Requirements

Design ESALs[1] (million)	Coarse Aggregate Angularity (Percent), minimum		Uncompacted Void Content of Fine Aggregate (Percent), minimum		Sand Equivalent (Percent), minimum	Flat and Elongated[3] (Percent), maximum
	Depth from Surface		Depth from Surface			
	≤ 100 mm	> 100 mm	≤ 100 mm	> 100 mm		
< 0.3	55/-	-/-	-	-	40	-
0.3 to < 3	75/-	50/-	40	40	40	10
3 to < 10	85/80[2]	60/-	45	40	45	
10 < 30	95/90	80/75	45	40	45	
≥ 30	100/100	100/100	45	45	50	

(1) Design ESALs are the anticipated project traffic level expected on the design lane over a 20-year period. Regardless of the actual design life of the roadway, determine the design ESALs for 20 years, and choose the appropriate N_{design} level.

(2) 85/80 denotes that 85 % of the coarse aggregate has one fractured face and 80 % has two or more fractured faces.

(3) Criterion based upon a 5:1 maximum-to-minimum ratio.

Note 5 - If less than 25% of a layer is within 100 mm of the surface, the layer may be considered to be below 100 mm for mixture design purposes.

Table: Simulation Study Test Results (ASTM D 5821), CAA

Stockpiles/Blends	1^+ Fractured	Criterion	2^+ Fractured	Criterion
Coarse Aggregate	99 %	__ % min	97 %	__ % min
Intermediate Aggregate	80 %		60 %	

This test is commonly only performed on the coarse aggregates during the initial screening of materials, even though the fine aggregate stockpiles may contain a small percentage retained on the 4.75 millimeter sieve. This test should also be run on the plus 4.75 millimeter material of the final design aggregate blend.

Q. Based on the table, what is the criterion for this surface mixture with an estimated traffic of 6,300,000 ESALs, (fill in the above table)?

Do both stockpiles meet the criteria, (Y/N)? If the answer is “no,” what does this mean?
(1) Stockpile cannot be used. or
(2) Percentage of stockpile in blend is limited.

Table: Simulation Study Test Results (AASHTO TP 33), FAA

Stockpiles/Blends	% Air Voids	Criterion
Manufactured Fines	48	
Natural Fines	42	≥ __

Q. Based on the table, what is the criterion for this surface mixture with an estimated traffic of 6,300,000 ESALs, (fill in the above table)?

Do both stockpiles meet the criteria, (Y/N)? If the answer is "no," what does this mean?
a. Stockpile cannot be used. or
b. Percentage of stockpile in blend is limited.

Author's Note

Fine aggregates with higher angularity may aid in the development of higher voids in mineral aggregate (VMA).

Table: Simulation Study Test Results (ASTM D 4791), F&E

Stockpiles/Blends	% Elongated	Criterion
Coarse Aggregate	9 %	__ %
Intermediate Agg.	2 %	

Q. Based on the table, what is the criterion for this surface mixture with an estimated traffic of 6,300,000 ESALs, (fill in the above table)?

Do both stockpiles meet the criteria, (Y/N)? If the answer is "no," what does this mean?

a. Stockpile cannot be used. or
b. Percentage of stockpile in blend is limited.

Table: Simulation Study Test Results (AASHTO T 176), SE

Stockpiles/Blends	Sand Equivalent	Criterion
Manufactured Fines	51 %	
Natural Fines	39 %	__ %
Intermediate Aggregate	45%	

Q. Based on the table, what is the criterion for this surface mixture with an estimated traffic of 6,300,000 ESALs, (fill in the above table)?

Do both stockpiles meet the criteria, (Y/N)? If the answer is "no," what does this mean?

a. Stockpile cannot be used. or
b. Percentage of stockpile in blend is limited.

Lead States Recommendations

Aggregate Consensus Properties - If Superpave criteria allow the use of aggregates with lower quality than previously used in a State, consideration should be given to maintaining the States' more stringent requirements until all Superpave validation work is complete. With respect to specific aggregate consensus properties, the following is offered:

Coarse Aggregate Angularity - Previous references in SHRP reports and elsewhere to the Pennsylvania Department of Transportation Test Method No. 621 for determining coarse aggregate angularity have been revised in AASHTO MP2, "Standard Specification for Superpave Volumetric Mix Design" to reference ASTM D5821, "Standard Test Method for Determining the Percentage of Fractured Particles in Coarse Aggregate," to more critically discriminate between aggregates.

Fine Aggregate Angularity - Fine aggregate angularity should be determined in accordance with AASHTO TP-33, "Uncompacted Void Content of Fine Aggregate," method A. The Lead States recommend the current Superpave fine aggregate angularity requirement of 45 at greater-than 3 million ESALs and 40 at less-than 3 million ESALs be specified. It should be noted that the aggregate's bulk specific gravity is a critical factor in the determination of the fine aggregate angularity, therefore, this value should be determined on a frequency appropriate for the variability of the source.

Flat-and-Elongated Particle Content - Excessive amounts of flat-and-elongated particles in a mixture can potentially lead to production and placement problems. This includes problems with volumetrics (both during design and production), aggregate degradation, and compaction.

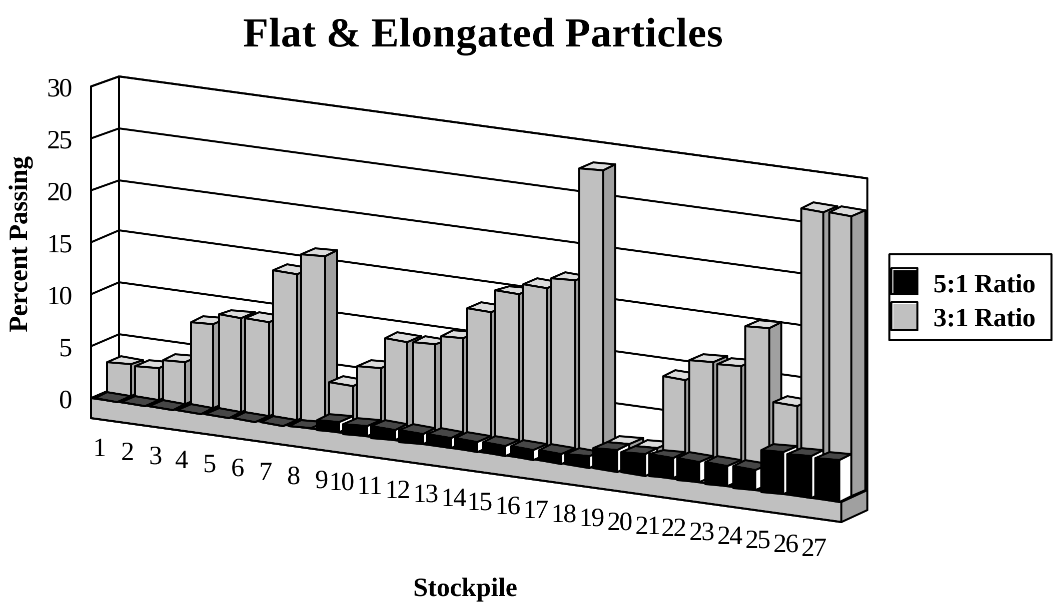
Current Superpave requirements (and other documentation) establish a 10% maximum flat-and-elongated particle content on material coarser than the 4.75 mm sieve when using a ratio of 5:1. This ratio is determined by comparing the maximum to minimum dimension. These dimensions should be visualized by circumscribed rectangular prisms around the aggregate. Testing is performed in accordance with ASTM D 4791, "Flat Particles, Elongated Particles, or Flat and Elongated Particles in Coarse Aggregate." It should be noted D 4791 requires testing to be performed on material coarser than the 9.5 mm sieve. Many believe testing aggregate passing the 9.5 mm sieve and retained on the 4.75 mm sieve will be very difficult and results highly variable. While this discrepancy is being addressed through AASHTO and ASTM, the Lead States recommend the states be aware of this issue and base specifications on their judgement of potential risks.

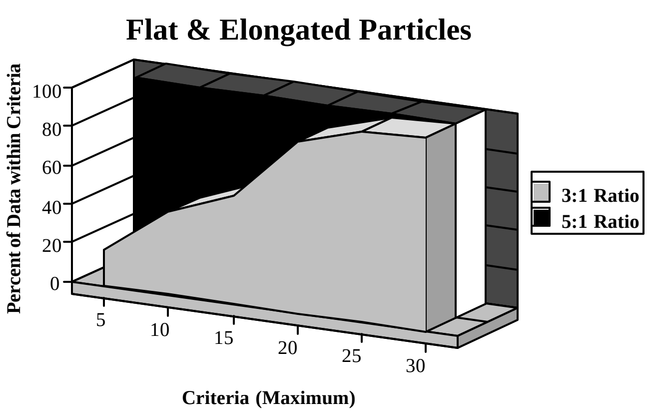
Many states have expressed concern that this criteria may not adequately discriminate between suitable and unsuitable aggregates and a 3:1 ratio should be specified. However, the relationship between flat-and-elongated particle content and performance has not been clearly established. There are currently several on-going research efforts attempting to establish this relationship.

Before changing the flat-and-elongated particle criteria to a 3:1 ratio, the Lead States recommend that past specifications and performance be considered. Further, until information is obtained relating flat-and-elongated particle content to performance, the maximum allowable value should not be set lower than 20%. This value is consistent with existing SMA criteria and has been used successfully in the past. Caution should be exercised when considering this change as it may significantly affect the use of certain materials which may otherwise prove to be suitable.

Mixture ETG Discussion

Under the auspicious of the Mixture expert task group, stockpile data collected as part of DP 90 was offered for discussion of the use of the 3:1 ratio. 27 Stockpiles from 12 different projects sites located in: California, Nevada, Alabama, Maine, Louisiana, Missouri, Illinois, South Carolina, Connecticut, Texas, Wisconsin, Minnesota, and Oklahoma.





The stockpile data is sorted above by increase F&E values.

The plot to the left shows the percent of these stockpile that would fail a range of criteria. In this case, all of the 27 stockpiles meet a 5:1 maximum criteria as tight as 5 % and 98% of the stockpiles meet a 3:1 maximum criteria of 25% .

It is recommended each specifying agency should perform a market analysis to access the impact of specifying a 3:1 source property standard.

SOURCE PROPERTY STANDARDS

Toughness as determined by:
L.A. Abrasion (AASHTO T 96)

Toughness is the percent loss of materials from an aggregate blend during the Los Angeles Abrasion test. The procedure is stated in AASHTO T 96, "Resistance to Abrasion of Small Size Coarse Aggregate by Use of the Los Angeles Machine." This test estimates the resistance of coarse aggregate to abrasion and mechanical degradation during handling, construction, and service. It is performed by subjecting the coarse aggregate, usually larger than 2.36 mm, to impact and grinding by steel spheres. The test result is percent loss, which is the weighted percentage of coarse material lost during the test as a result of the mechanical degradation.

Maximum allowable loss values typically range from approximately 35 to 45 percent.

Soundness as determined by:
Sulfate Soundness (AASHTO T 104)

Soundness is the percent loss of materials from an aggregate blend during the sodium or magnesium sulfate soundness test. The procedure is stated in AASHTO T 104, "Soundness of Aggregate by Use of Sodium Sulfate or Magnesium Sulfate." This test estimates the resistance of aggregate to weathering while in-service. It can be performed on both coarse and fine aggregate. The test is performed by alternately exposing an aggregate sample to repeated immersions in saturated solutions of sodium or magnesium sulfate each followed by oven drying. One immersion and drying is considered one soundness cycle. During the drying phase, salts precipitate in the permeable void space of the aggregate. Upon re-immersion the salt re-hydrates and exerts internal expansive forces that simulate the expansive forces of freezing water. The test result is total percent loss over various sieve intervals for a required number of cycles.

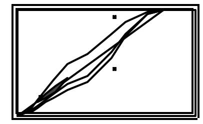
Maximum allowable loss values typically range from approximately 10 to 20 percent for five cycles.

Deleterious Material as determined by: Clay Lumps and Friable Particles (AASHTO T 112)

Deleterious materials are defined as the weight percentage of contaminants such as shale, wood, mica, and coal in the blended aggregate. This property is measured by AASHTO T 112, "Clay Lumps and Friable Particles in Aggregates." It can be performed on both coarse and fine aggregate. The test is performed by wet sieving aggregate size fractions over prescribed sieves. The weight percentage of material lost as a result of wet sieving is reported as the percent of clay lumps and friable particles.

A wide range of maximum permissible percentage of clay lumps and friable particles is evident. Values range from as little as 0.2 percent to as high as 10 percent, depending on the exact composition of the contaminant.

② SELECTION OF A DESIGN AGGREGATE STRUCTURE



The FHWA 0.45 Power gradation chart is used to define permissible gradations. This chart uses a unique graphing technique to judge the cumulative particle size distribution of a blend. The ordinate (y axis) of the chart is percent passing. The abscissa (x axis) is an arithmetic scale of sieve size opening in microns, raised to the 0.45 power.

To select the design aggregate structure, trial blends are established by mathematically combining the gradations of the individual materials into a single blend. The blend is then compared to the specification requirements for the appropriate sieves. Gradation control is based on four control sieves: the maximum sieve, the nominal maximum sieve, the 2.36 mm sieve, and the 0.075 mm sieve. Definitions:

- *Nominal Maximum Sieve Size:* One standard sieve size larger than the first sieve to retain more than 10 percent.

- *Maximum Sieve Size:* One standard sieve size larger than the nominal maximum size. The 0.45 power maximum density line is draw from the origin to 100 percent passing the maximum size.

Standard Sieves
50.0 mm
37.5 mm
25.0 mm
19.0 mm
12.5 mm
9.50 mm
4.75 mm
2.36 mm
1.18 mm
0.60 mm
0.30 mm
0.15 mm

0.075 mm

Q. Match the English Sieves to their Standard equivalents:

English Sieves				Standard Sieves
(1)	No. 100		(A)	50.0 mm
(2)	No. 4		(B)	37.5 mm
(3)	1/4 inch		(C)	25.0 mm
(4)	1 inch		(D)	19.0 mm
(5)	No. 200		(E)	12.5 mm
(6)	No. 80		(F)	9.50 mm
(7)	No. 50		(G)	4.75 mm
(8)	½ inch		(H)	2.36 mm
(9)	No. 16		(I)	1.18 mm
(10)	No. 20		(J)	0.60 mm
(11)	No. 40		(K)	0.30 mm
			(L)	0.15 mm
			(M)	0.075 mm

A. (1)L, (2)G, (3)*, (4)C, (5)M, (6)*, (7)K, (8)E, (9)I, (10)*, (11)*

* - English sieve is not part of the Standard sieve stack.

There is also a recommended *"restricted zone."* The restricted zone is an area on either side of the maximum density line generally starting at the 2.36 millimeter sieve and extending to the 0.300 millimeter sieve. The minimum and maximum values required for the control sieves change (as does the restricted zone) as the nominal size of the blend changes. The following table defines the control points and recommended restricted zones for different nominal maximum sieve sizes.

Table: Superpave Aggregate Gradation Requirements

Standard Sieve (mm)	**Percent Passing Criteria *(Control Points)***				
	Nominal Maximum Sieve Size				
	9.5 mm	**12.5 mm**	**19 mm**	**25 mm**	**37.5 mm**
50.0					100
37.5				100	90 - 100
25.0			100	90 - 100	
19.0		100	90 - 100		
12.0	100	90 - 100			
9.50	90 - 100				
2.36	32 - 67	28 - 58	23 - 49	19 - 45	15 - 41
0.075	2.0 - 10.0	2.0 - 10.0	2.0 - 8.0	1.0 - 7.0	0.0 - 6.0
Sieve	**Recommended Restricted Zone**				
4.75				39.5	34.7
2.36	47.2	39.1	34.6	26.8 - 30 8	23.3 - 27.3
1.18	31.6 - 37.6	25.6 - 31.6	22.3 - 28.3	18.1 - 24.1	15.5 - 21.5
0.60	23.5 - 27.5	19.1 - 23.1	16.7 - 20.7	13.6 - 17.6	11.7 - 15.7
0.30	18.7	15.5	13.7	11.4	10.0

All trial blend gradations *(washed in accordance to AASHTO T-11)* must pass between the control points established. In addition, they should be outside of the area bounded by the limits set for the restricted zone.

Restricted Zone - NCHRP 9-14, entitled, *"Investigation of the Restricted Zone in the Superpave Aggregate Gradation Specification,"* researched the impact of mixes crossing and outside of the restricted zone. The research supports the elimination of the restricted zone as long as ALL other Superpave design criteria is satisfied - specifically FAA and volumetrics. The work was conducted by the National Center for Asphalt Technology. The final report was completed in April 2001.

Typically the State highway agency will specify the nominal maximum size required for the pavement layer. For our simulation study, the specified size is 19.0 mm. It is recommended that three trial blends be initially developed.

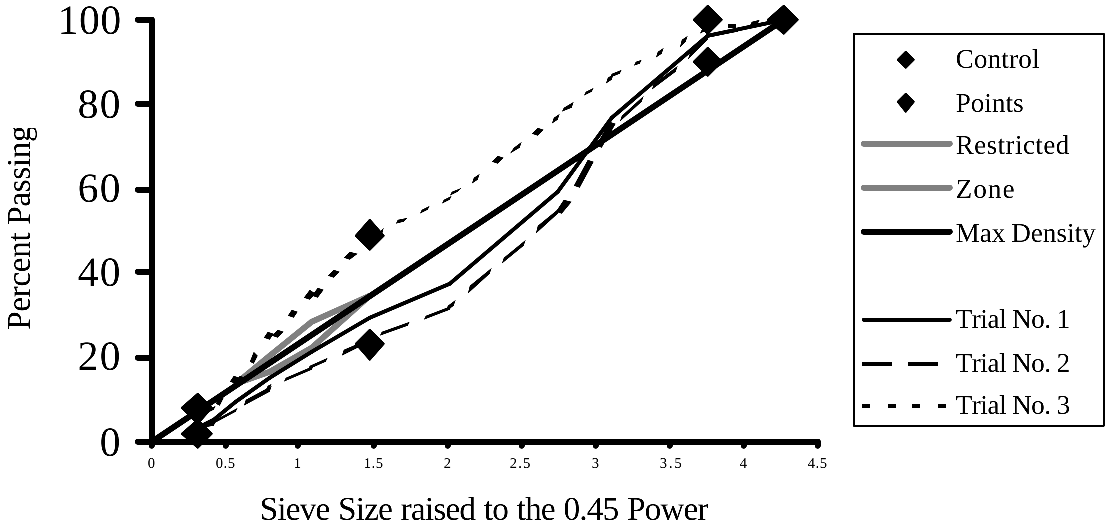
Table: Develop Trial Blends

	Mineral Aggregates				Trial No. 1	Trial No. 2	Trial No. 3
	Coarse Agg.	Intr. Agg.	Man. Fines	Natr'l. Fines	46% 24% 15% 15%	51% 25% 15% 9%	25% 24% 23% 28%
Sieve	Stockpile Gradations				No. 1	No. 2	No. 3
37.5 mm	100.0	100.0	100.0	100.0	100.0	100.0	100.0
25.0 mm	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0 mm	92.0	100.0	100.0	100.0	96.3	95.9	98.0
12.5 mm	50.0	100.0	100.0	100.0	77.0	74.5	87.5
9.5 mm	14.0	95.0	100.0	100.0	59.2	54.9	77.3
4.75 mm	3.0	25.0	100.0	100.0	37.4	31.8	57.8
2.36 mm	2.0	6.0	87.0	93.0	29.4	23.9	48.0
1.18 mm	2.0	4.0	65.0	64.0	21.2	17.5	34.3
0.60 mm	2.0	4.0	42.0	48.0	15.4	12.6	24.6
0.30 mm	2.0	3.0	18.0	32.0	9.1	7.4	14.3
0.15 mm	2.0	3.0	6.0	18.0	5.2	4.3	7.6
0.075 mm	2.0	2.8	3.7	10.0	3.6	3.2	5.8

! **Nominal Maximum Sieve Size:** One standard sieve size larger than the first sieve to retain more than 10 percent. The first sieve to retain more than 10 percent for all blends is the 12.5 millimeter. On sieve larger is the 19.0 millimeter. Such that the nominal maximum sieve size is the 19.0 millimeter.

! **Maximum Sieve Size:** One standard sieve size larger than the nominal maximum size. Such that the 25.0 millimeter is the maximum sieve size.

Figure: Trial Blends 0.45 Power Chart



Once the trial blends are established, preliminary determinations of the blended aggregate properties can be determined. This can be estimated mathematically from the individual aggregate properties using the blend percentages. The combined aggregate bulk and apparent specific gravities are determined using the law of partial fractions. *(If the individual properties were not previously determined, the consensus and source properties standards need to be determined for the design aggregate blend.) Example:*

Stockpile	**Trial Blend #1 Percentage**	**Test Results Bulk Sp.Gv. (G_{sb})**
Coarse Aggregate	46 %	2.567
Intermediate Agg	24 %	2.587
Manufactured Fines	15 %	2.501
Natural Fines	15 %	2.598

Estimated Trial Blend #1 G_{sb}

$$G_{sb} = \frac{P_1 + P_2 + \ldots + P_n}{\left(\frac{P_1}{G_1} + \frac{P_2}{G_2} + \ldots + \frac{P_n}{G_n}\right)} = \frac{100}{\left(\frac{46}{2.567} + \frac{24}{2.587} + \frac{15}{2.501} + \frac{15}{2.598}\right)} = 2.566$$

where: G_{sb} = bulk specific gravity for the total aggregate blend
P_1 , P_2 ,...P_n = percentage by weight of aggregates, 1, 2,...n

G_1 , G_2 ,...G_n = bulk specific gravity of aggregates, 1, 2,...n

For the estimation of the consensus and source property standards a method developed by Gerry Huber (Heritage Research Group) is used:

Huber's Method:

Step 1: First the portion of each stockpile that applies to a property standard is determined for each trial blend. This is simply the percentage of the stockpile used in the trial blend multiplied by the percentage of the stockpile which applies to property standard (ex. For the coarse aggregate angularity it would be the plus 4.75 mm material).

Trail Blend No. 1: Coarse Aggregate Angularity (CAA)

Stockpile	CAA Test Result	(A) Trial Blend	(B) + 4.75 mm	D = (A x B) % App. To CAA
Coarse	99/97	46 %	97 %	44.6 %
Intermediate	80/60	24 %	75 %	18 %
Man. Fines	/	15 %	0 %	0
Natural Fines	/	15 %	0 %	0

Note: % App. To CAA is the portion of the stockpiles that apply to the consensus property.

Step 2: Determine the estimated property:

$$\text{Est. Pr operty} = \frac{[(CxD)1 + (CxD)2 \ldots n]}{[(D)1 + (D)2 \ldots n]}$$

where: Est. Property is the consensus or source property
C = Test Result
D = Portion of the stockpile that applies to consensus or source property
n = Stockpile number

$$CAA_{+1} = \frac{[(99x44.2) + (80x14.4)]}{[(44.2) + (14.4)]} = 94\,\%$$

Table: Huber's Method - Trial Blend No. 1

Stock-pile	(A) Trial Blend	(B)				D = (A x B)			
		CAA +4.75	FAA -2.36	F/E +9.5	SE -4.75	CAA	FAA	F/E	SE
Coarse	46 %	97	2	86	3	44.6	- (*)	39.6	-
Inter.	24 %	75	6	5	25	18.0	-	-	6.0
Man.F.	15 %	0	87	0	100	-	13.1	-	15.0
Nat. F.	15 %	0	93	0	100	-	14.0	-	15.0

(*) The stockpile is not considered if less than 10 % of the stockpile applies to the property standard.

Table: Huber's Method - Trial Blend No. 2

Stock-pile	(A) Trial Blend	(B)				D = (A x B)			
		CAA +4.75	FAA -2.36	F/E +9.5	SE -4.75	CAA	FAA	F/E	SE
Coarse	51 %	97	2	86	3	49.5	-	43.9	-
Inter.	25 %	75	6	5	25	18.8	-	-	6.3
Man.F.	15 %	0	87	0	100	-	13.1	-	15.0
Nat. F.	9 %	0	93	0	100	-	8.4	-	9.0

Table: Huber's Method - Trial Blend No. 3

Stock-pile	(A) Trial Blend	(B)				D = (A x B)			
		CAA +4.75	FAA -2.36	F/E +9.5	SE -4.75	CAA	FAA	F/E	SE
Coarse	25 %	97	2	86	3	24.3	-	21.5	-
Inter.	24 %	75	6	5	25	18.0	-	-	6.0
Man.F.	23 %	0	87	0	100	-	20.0	-	23.0
Nat. F.	28 %	0	93	0	100	-	26.0	-	28.0

Table: Summary of Actual Stockpile and Estimated Blend Properties

Property	Criteria	Actual Test Values (C)				Calculated Values		
		Stockpile A	**Stockpile B**	**Stockpile C**	**Stockpile D**	**Trial Blend No.1**	**Trial Blend No. 2**	**Trial Blend No. 3**
CAA	85/80min	99/97	80/60	n/a	n/a	94/86	94/87	91/____?
FAA	45 min	n/a	n/a	48	42	45	46	45
F & E	10 max	9	2	n/a	n/a	9	9	9
SE	45 min	n/a	45	51	39	45	46	_____?
Bulk Sp.Gv., G_{sb}		2.567	2.587	2.501	2.598	2.566	2.565	2.565
Apparent Sp.Gv., G_{sa}		2.680	2.724	2.650	2.673	2.685	2.686	2.681

Q. Complete the above table for Trial Blend No. 3 . . .

A. CAA2$^+$ = (97 * 24.3 + 60 *18.0) / (24.3 + 18.0) = 81
SE = (45*6 + 51*23 + 39*28) / (6 + 23 + 28) = 44

Estimate Trial Blend Asphalt Binder Contents

The next step is to evaluate the trial blends by compacting specimens and determining the volumetric properties of each trial blend. The trial asphalt binder content can be determined for each trial blend by estimating the effective specific gravity (G_{se}) of the blends and using the calculations shown below. This estimate is based on several assumptions that may or may not apply to local aggregates. It is important to approximate the trial asphalt binder content from experience prior to performing the calculations.

Based on experience for 19 millimeter nominal, surface course mixture the asphalt content should be approximately...

___?___ %

<u>Estimate Trial Blend Asphalt Contents - Calculations</u>:

Calculations for estimating the trial asphalt binder content can be divided into four steps.

Step 1: Estimate aggregate effective specific gravity
Step 2: Estimate volume of absorbed binder
Step 3: Estimate volume of effective binder
Step 4: Estimate trial binder content

Step 1: Estimate the effective specific gravity (G_{se}) of the trial blends:

$$G_{se} = G_{sb} + 0.8 * (G_{sa} - G_{sb})$$

0.8 factor accounts for absorption, high absorptive aggregates may require values closer to <u>0.6 or 0.5</u>. For this example, *experience* with this equation and local aggregates dictates a factor of 0.6.

Trial Blend No. 1; G_{se} = 2.566 + 0.6 * (2.685 - 2.566) = 2.637

Table: Estimated Effective Specific Gravities

Trial Blend	G_{se}
Trial Blend No. 1	2.637
Trial Blend No. 2	2.637
Trial Blend No. 3	2.635

Step 2: Estimate the volume of asphalt binder (V_{ba}) absorbed into the aggregate:

$$V_{ba} = \frac{P_s * (1 - V_a)}{\left(\frac{P_b}{G_b} + \frac{P_s}{G_{se}}\right)} * \left(\frac{1}{G_{sb}} - \frac{1}{G_{se}}\right)$$

V_{ba}	=	volume of absorbed binder	
V_a	=	volume of air voids	= 0.04
P_b	=	percent of binder	≈ 0.05
P_s	=	percent of aggregate	≈ 0.95
G_b	=	spec grav of binder	= 1.030

$$\textit{Trial Blend No. 1;}\ V_{ba} = \frac{0.95 * (1 - 0.04)}{\left(\frac{0.05}{1.030} + \frac{0.95}{2.637}\right)} * \left(\frac{1}{2.566} - \frac{1}{2.637}\right) = 0.0233$$

Table: Estimated Volume of Absorbed Binder

Trial Blend	V_{ba}
Trial Blend No. 1	0.0233
Trial Blend No. 2	0.0239
Trial Blend No. 3	0.0232

Step 3: Estimate the volume of effective binder (V_{be}) of the trial blends:

$V_{be} = 0.176 - 0.0675 * \text{Logarithm}_{natural} (S_n)$

where:

S_n = the nominal maximum sieve size of the aggregate blend in millimeters

V_{be} = 0.176 - 0.0675 * Ln (19.0) = <u>0.090</u> (for all blends)

Table: Estimated Volume of Effective Binder

Nominal Max, S_n	**V_{be}**
50.0 mm	0.061
37.5 mm	0.070
25.0 mm	0.082
19.0 mm	0.090
12.5 mm	0.102
9.50 mm	0.110
4.75 mm	0.130

Step 4: Estimate initial trial asphalt binder (P_{bi}) content for the trial blends:

$$W_s = \frac{P_s * (1 - V_a)}{(P_b/G_b + P_s/G_{se})}$$

$$P_{bi} = \frac{G_b * (V_{be} + V_{ba})}{(G_b * (V_{be} + V_{ba})) + W_s} * 100$$

where:

W_s = weight of aggregate
P_{bi} = percent (by weight of mix) of binder

$$\text{Trial Blend No. 1; } W_s = \frac{0.95 * (1 - 0.04)}{(0.05/1.030 + 0.95/2.637)} = 2.231$$

$$\text{Trial Blend No. 1; } P_{bi} = \frac{1.030 * (0.090 + 0.0233)}{(1.030 * (0.090 + 0.0233)) + 2.231} * 100 = 4.95\ \%$$

Table: Estimated Weight of Aggregate and Percent of Binder

Trial Blend	**W_s**	**P_{bi}**
Trial Blend No. 1	2.231	4.95 %
Trial Blend No. 2	2.231	4.98 %
Trial Blend No. 3	2.229	4.95 %

Author's Note

The estimated percent of binder determined by the equations can not replace experience. Local aggregate and binders when combined will almost always require a slightly different optimum asphalt content. Your experience with Superpave mixtures should always govern over these calculated estimates.

Next: Evaluate Trial Blends at Estimated Asphalt Binder Contents

Table: Required Tests

Trial Blend	Superpave Gyratory Compactor Specimens	Rice, G_{mm} Max Specific Gravity (T 209)
Number 1	3 Specimens *4800 g/ea*	2 Tests *2000 g/ea*
Number 2	3 Specimens	2 Tests
Number 3	3 Specimens	2 Tests
Total *(55,200 g)*	9 Specimens *(43,200 g)*	6 Tests *(12,000 g)*

A minimum of two specimens (FHWA recommends three) for each trial blend are compacted using the Superpave gyratory compactor. A mixture weight of 4800 grams is usually sufficient for the compacted specimens. Two specimens are also prepared for determination of the mixture's maximum theoretical specific gravity, (G_{mm}). A mixture weight of 2000 grams is usually sufficient for the specimens used to determine G_{mm}. Excerpt, AASHTO T 209:

Nominal Maximum Size of Aggregate (mm)	Minimum Mass of Sample (kg)
25.0	2.5
19.0	2.0
12.5	1.5
9.5	1.0
4.75	0.5

Author's Note

Nominal maximum size of aggregate for the above table is based on AASHTO definition not Superpave. Such that the nominal maximum size is the smallest sieve size through which the entire amount of aggregate is permitted to pass. How does this relate to Superpave?

Aging

Specimens are mixed at the appropriate mixing temperature based on the temperature-viscosity relationship. The specimens are short-term aged. The original procedure required 4 hours of short term aging in a forced-draft oven at 135˚C. The mix is spread to a density of 21 to 22 kilograms per square meter (kg/m²) of pan (approximately 10 mm thick). The specimens are hand mixed every hour. The Lead States propose an alternate procedure, based on the following rationale:

Lead States' Rationale

NCHRP 9-9, "Evaluation of the Superpave Gyratory Compaction Procedure," research performed by NCAT has shown there is not a practical difference for non-absorptive aggregates in mixture volumetric properties when 2- or 4-hour conditioning is performed. This research confirmed previous findings of the Mixture Expert Task Group. Additionally, NCAT evaluated the difference in a mixture's volumetric properties when aging is performed at the mixture's compaction temperature and aging at 135 °C. While differences were noted, it was determined that these differences were inconsequential from an engineering perspective. However, additional research sited by the FHWA indicates there is a difference in the resulting mechanical properties of mixtures conditioned for 2 versus 4 hours. Adopting a specific 2-hour mixture conditioning period for the volumetric mixture design procedure at the mixture's compaction temperature will expedite mixture design development. The existing "short and long" aging procedures are maintained for use when mechanical property testing of the mixture will be performed.

Summary of Practice

Original: For short term aging a mixture of aggregate and asphalt binder is aged in a forced- draft oven for 4 hours at 135°C. For long term aging a compacted mixture of aggregate and asphalt binder is aged in a forced-draft oven for 5 days at 85°C.

Current '99: For mixture conditioning for <u>volumetric mixture design</u>, a mixture of aggregate and asphalt binder is conditioned in a forced-draft oven for 2 hours at the mixture's specified compaction temperature.

For short-term mixture conditioning for mechanical property testing, a mixture of aggregate and asphalt binder is aged in a forced-draft oven for 4 hours at 135°C.

For long-term mixture conditioning for mechanical property testing, a compacted mixture of aggregate and asphalt binder is aged in a forced-draft oven for 5 days at 85°C.

<u>Compaction</u>

History Lesson

The Superpave system, developed under SHRP, employs gyratory compaction to fabricate asphalt mixture specimens. The level of compaction in the SGC is based upon the design traffic and the average 7-day maximum air temperature. The design traffic is the projected, single lane, traffic volume over 20 years - expressed in ESALs. AASHTO MP-2 provides a table for selection of specimen compaction levels. The table has seven traffic categories and four ranges of temperatures, constituting a total matrix of twenty-eight (28) different compaction levels.

The compaction table is based on research conducted under the SHRP contract by the Asphalt Institute, (AI). The researchers evaluated 9, in-service, general pavement studies, *(GPS)*, from across the United States, using a prototype gyratory compactor. All of the GPS sites were performing well after several years of service. The sites were cored and volumetrics were determined. Aggregates were then recovered and recombined with a standard asphalt binder (AC-20) and compacted in a prototype SGC. The compaction efforts required to produce four percent air voids were determined. This effort was then equated to traffic level and site environmental data resulting in the table of compaction levels.

NCHRP 9-9 entitled, "<u>Refinement of the Superpave Gyratory Compaction Procedure</u>", conducted by NCAT, evaluated the sensitivity of the compaction levels. The principal investigator, Dr. E. Ray Brown, and his team investigated whether there is any significant volumetric property differences between mixtures compacted at the various compaction levels.

A parallel effort conducted by the FHWA Mixture ETG, investigated the validity of the number of gyrations used to design asphalt mixtures. This effort, designated, "N-design II," was conducted through the AI in partnership with Heritage Research Group. The principal researchers included Mike Anderson *(AI)*, Gerry Huber *(Heritage)*, Bob McGennis *(South Central Superpave Regional Center)*, and Rich May *(previously with AI, now with Koch Materials)*. The researchers were provided with samples and data from several State Highway Agencies, FHWA Turner Fairbank Highway Research Center, and FHWA Performance Related Specifications Test Track, (WesTrack).

The NCHRP 9-9 research effort developed a simplified, compaction matrix. As did the research N-design II effort. During the Mix ETG meeting held September 22 and 23, 1998 in Baltimore, Maryland, the expert task group reviewed the findings of both research efforts. On September 24, 1998, the Superpave Lead States met and concurred with the efforts of the Mix ETG. These efforts resulted in the development of a new compaction matrix. The new compaction table was forwarded by the Mix ETG to AASHTO for balloting and inclusion in the standards.

The Superpave compaction criteria are based on three points during the compaction effort: an initial (N_{ini}), design (N_{des}), and maximum (N_{max}) number of gyrations. Limiting criteria based on the percent of G_{mm} has also been established for the initial, design, and maximum number of gyrations.

PP-28, Table 1 - Superpave Gyratory Compaction Effort

Design ESALs[1] (million)	Compaction Parameters			Typical Roadway Application[2]
	$N_{initial}$	N_{design}	N_{max}	
< 0.3	6	50	75	Applications include roadways with very light traffic volumes such as local roads, county roads, and city streets where truck traffic is prohibited or at a very minimal level. Traffic on these roadways would be considered local in nature, not regional, intrastate, or interstate. Special purpose roadways serving recreational sites or areas may also be applicable to this level.
0.3 to < 3	7	75	115	Applications include many collector roads or access streets. Medium-trafficked city streets and the majority of county roadways may be applicable to this level.
3 to < 30	8	100	160	Applications include many two-lane, multilane, divided, and partially or completely controlled access roadways. Among these are medium-to highly-trafficked city streets, many state routes, US highways, and some rural interstates.
≥ 30	9	125	205	Applications include the vast majority of the US Interstate system, both rural and urban in nature. Special applications such as truck-weighing stations or truck-climbing lanes on two-lane roadways may also be applicable to this level.

(1) Design ESALs are the anticipated project traffic level expected on the design lane over a 20-year period. Regardless of the actual design life of the roadway, determine the design ESALs for 20 years, and choose the appropriate N_{design} level.

(2) Typical Roadway Applications as defined by A Policy on Geometric Design of Highway and Streets, 1994, AASHTO.

Note 17 -- When specified by the agency and the top of the design layer is ≥ 100 mm from the pavement surface and the estimated design traffic level ≥ 0.3 million ESALs, decrease the estimated design traffic level by one, unless the mixture will be exposed to significant main line and construction traffic prior to being overlaid. If less than 25% of the layer is within 100 mm of the surface, the layer may be considered to be below 100 mm for mixture design purposes.

Note 18 – When the design ESALs are between 3 to < 10 million ESALs the agency may, at their discretion, specify $N_{initial}$ at 7, N_{design} at 75, and N_{max} at 115, based on local experience.

PP-35, Table 2 - Superpave Volumetric Mixture Design Requirements

<table>
<tr><th rowspan="3">Design ESALs[1] (million)</th><th colspan="3">Required Density (% of Theoretical Maximum Specific Gravity)</th><th colspan="5">Voids-in-the Mineral Aggregate (Percent), minimum</th><th rowspan="3">Voids Filled With Asphalt (Percent)</th><th rowspan="3">Dust-to-Binder Ratio</th></tr>
<tr><th rowspan="2">$N_{initial}$</th><th rowspan="2">N_{design}</th><th rowspan="2">N_{max}</th><th colspan="5">Nominal Maximum Aggregate Size, mm</th></tr>
<tr><th>37.5</th><th>25.0</th><th>19.0</th><th>12.5</th><th>9.5</th></tr>
<tr><td>< 0.3</td><td>≤ 91.5</td><td rowspan="5">96.0</td><td rowspan="5">≤ 98.0</td><td rowspan="5">11.0</td><td rowspan="5">12.0</td><td rowspan="5">13.0</td><td rowspan="5">14.0</td><td rowspan="5">15.0</td><td>70 - 80 [3,4]</td><td rowspan="5">0.6 - 1.2</td></tr>
<tr><td>0.3 to < 3</td><td>≤ 90.5</td><td>65 - 78[4]</td></tr>
<tr><td>3 to < 10</td><td rowspan="3">≤ 89.0</td><td rowspan="3">65 - 75[2,4]</td></tr>
<tr><td>10 to < 30</td></tr>
<tr><td>≥ 30</td></tr>
</table>

(1) Design ESALs are the anticipated project traffic level expected on the design lane over a 20-year period. Regardless of the actual design life of the roadway, determine the design ESALs for 20 years, and choose the appropriate N_{design} level.

(2) For 9.5-mm nominal maximum size mixtures, the specified VFA range shall be 73% to 76% for design traffic levels ≥3 million ESALs.

(3) For 25.0-mm nominal maximum size mixtures, the specified lower limit of the VFA shall be 67% for design traffic levels < 0.3 million ESALs.

(4) For 37.5-mm nominal maximum size mixtures, the specified lower limit of the VFA shall be 64% for all design traffic levels.

Note 19 -- If the aggregate gradation passes beneath the boundaries of the aggregate restricted zone specified in Table 3, consideration should be given to increasing the dust-to-binder ratio criteria from 0.6 - 1.2 to 0.8 - 1.6.

PP-28, Table 4 - Selection of a Design Aggregate Structure (Example)

Volumetric Property	Trial Mixture (19.0 mm nominal maximum aggregate) 20-Year Project Design ESALs = 5 million			Criteria	
	1	2	3		
	At the initial trial asphalt content				
P_b (trial)	4.4	4.4	4.4		
$\%Gmm_{initial}$ (trial)	88.1	88.8	87.1		
$\%Gmm_{design}$ (trial)	95.9	95.3	94.7		
V_a at N_{design}	4.1	4.7	5.3	4.0	
VMA_{trial}	12.9	13.4	13.9		
	Adjustments to reach design asphalt content (V_a = 4.0 % at N_{design})				
ΔV_a	-0.1	-0.7	-1.3		
ΔP_b	0.0	0.3	0.5		
Δ VMA	0.0	-0.1	-0.3		
	At the estimated design asphalt content (V_a = 4.0 % at N_{design})				
Estimated P_b (design)	4.4	4.7	4.9		
VMA (design)	12.9	13.3	13.6	> 13.0	
$\%Gmm_{initial}$ (design)	88.2	89.5	88.4	ESALs $<0.3 \times 10^6$ $0.3 - 3 \times 10^6$ $> 3 \times 10^6$	Criteria, $\%G_{mm}$ <91.5 <90.5 < 89.0

Notes:

The top portion of this table presents measured compaction densities and volumetric properties for specimens prepared for each trial aggregate gradation at the initial trial asphalt content.

None of the specimens had an air void content of exactly 4.0 percent. Therefore, the procedures described in Section 9 must be applied to: 1) estimate the design asphalt content at which V_a = 4.0 percent, and 2) obtain adjusted VMA and density values at this estimated asphalt content.

The middle portion of this table presents the change in asphalt content (ΔP_b) and VMA (ΔVMA) that occurs when the air void content (V_a) is adjusted to 4.0 percent for each trial aggregate gradation.

A comparison of the VMA and densities at the estimated design asphalt content to the criteria in the last column shows that trial gradation #1 does not have sufficient VMA (12.9% versus a requirement of $\geq$ 13.0%). Trial gradation #2 exceeds the criterion for density at $N_{initial}$ gyrations (89.5 versus a requirement of < 89.0 %). Trial gradation #3 meets the requirements for density and VMA and, in this example, is selected as the design aggregate structure.

For Hot Mix, USA, the estimated, 20-year, design traffic is 6,300,000 ESALs. The traffic level falls in the 3 to less than 30 million ESAL range. The project is a State route, which falls in the typical roadway application defined in the current '99 table above. Such that, from the table the initial, design, and maximum number of gyrations are 8, 100, and 160, respectively. The following table summarizes the volumetric criteria for the project:

Table: Summary of Project Volumetric Criteria

Volumetric Property	Volumetric Criteria
N_{ini} $\%G_{mm}$ at N_{ini}	8 gyrations $\leq$89 %
N_{ini} $\%G_{mm}$ at N_{design}	100 gyrations = 96 % (4% air voids)
N_{max} $\%G_{mm}$ at N_{max}	160 gyrations $\leq$ 98 %
Voids-in-the Mineral Aggregate (VMA)	13.0 minimum
Voids Filled with Asphalt (VFA)	65 - 75 percent
Dust-to-Binder Ratio	0.6 - 1.2

For the evaluation of the trial blends, specimens are compacted to the **design** number of gyrations, with the specimen height collected during the compaction process. Since the specimen mass and cross section are constant throughout compaction, the density can be continually calculated based on the height.

After compaction is complete, the specimen is extruded and the bulk specific gravity is determined (G_{mb}) by AASHTO T 166. The G_{mm} of each blend is also determined by AASHTO T-209. From this, the design percent of maximum theoretical specific gravity ($\%G_{mm\ des}$) can be calculated. Such that, from the compaction height data (h_x), the $\%G_{mm}$ per gyration can be determined as follows:

$$Design,\ \%G_{mm\ des} = \frac{G_{mb}}{G_{mm}} * 100$$

Initial: $\%G_{mm\ ini} = \%G_{mm\ des} * (h_{des} / h_{ini})$

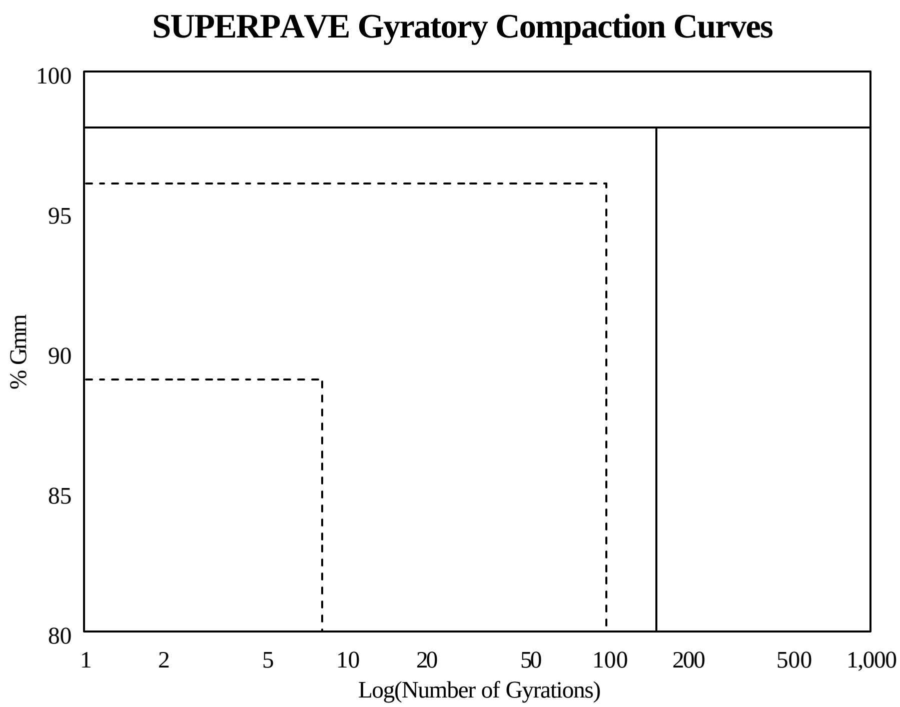


Figure: Gyratory Compaction Graph

Gyratory Compaction Calculations

For each option blend, three gyratory specimens are compacted (AASHTO TP 4) in the Superpave gyratory compactor to N_{des} and two maximum theoretical specific gravities are determined (AASHTO T 209) (G_{mm}). The gyratory specimens are extruded from the molds and bulk specific gravities are determined (G_{mb}).

<u>Trial Blend No. 1: Measured Properties of the Specimens</u>

G_{mm} = 2.475 *(Rice)*

Specimen 1: G_{mb} = 2.351 *(at design number of gyrations, N_{des})*
Specimen 2: G_{mb} = 2.348
Specimen 3: G_{mb} = 2.353

The percent of maximum theoretical specific gravity at N_{des} (% $G_{mm\ des}$) is calculated as follows:

$$\textit{Specimen 1: } \%G_{mm\ des} = \frac{G_{mb}}{G_{mm}} * 100 = \frac{2.351}{2.475} * 100 = 95.0\ \% = 96.0\ \%\ \textit{Criterion}$$

Q. What is $\%G_{mm\ des}$ for specimens 2 and 3 at N_{des} ?

Specimen 2: $\%G_{mm\ des}$ =))))) *100 = ___.__ %

Specimen 3: $\%G_{mm\ des}$ =))))) * 100 = ___.__ %

Table: Trial Blend No. 1: Specimen Compaction & Height Data

Trial Blend No. 1	Height N_{ini}=8	Height N_{des}=100	Height N_{max}=160	$\%G_{mm\ des}$
Specimen 1	129.6 mm	117.4 mm		95.0 %
Specimen 2	129.8 mm	117.4 mm	n/a	94.9 %
Specimen 3	129.9 mm	117.8 mm		95.1 %

As stated above, the initial $\%G_{mm}$ is calculated based on the height ratios multiplied by the design $\%G_{mm}$. Such that:

Specimen 1: $\%G_{mm\ ini} = \%G_{mm\ des} * (117.4 / 129.6) = 86.1\ \% \leq$ *89 % Criterion*

Q. What are the $\%G_{mm\ ini}$ for specimens 2 and 3 at N_{ini}?

Specimen 2: $\%G_{mm\ ini}$ = ___.__ % * (____.__/____.__) = ____.__ %

Specimen 3: $\%G_{mm\ ini}$ = ___.__ % * (____.__/____.__) = ____.__ %

Table: Trial Blend No. 1 Compaction Results

Specimen	$\%G_{mm\ ini}$ N_{ini}=8	$\%G_{mm\ des}$ N_{des}=100	$\%G_{mm\ max}$ N_{max}=160
1	86.1 %	95.0 %	
2	__._ %	94.9 %	n/a
3	__._ %	95.1 %	

Graph the results on the Superpave Gyratory Compaction Chart provided (See next page).

SUPERPAVE Gyratory Compactiom Chart

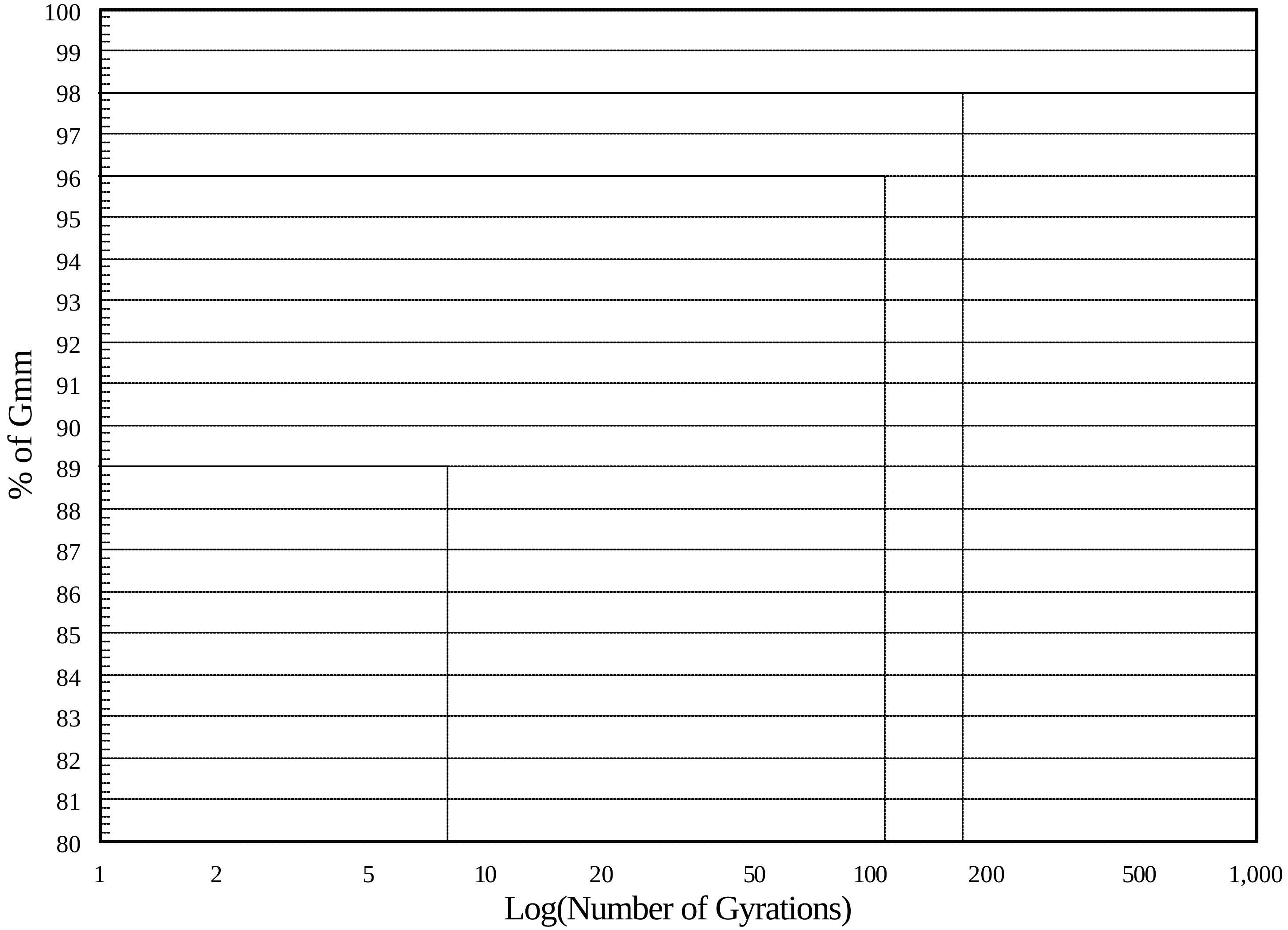
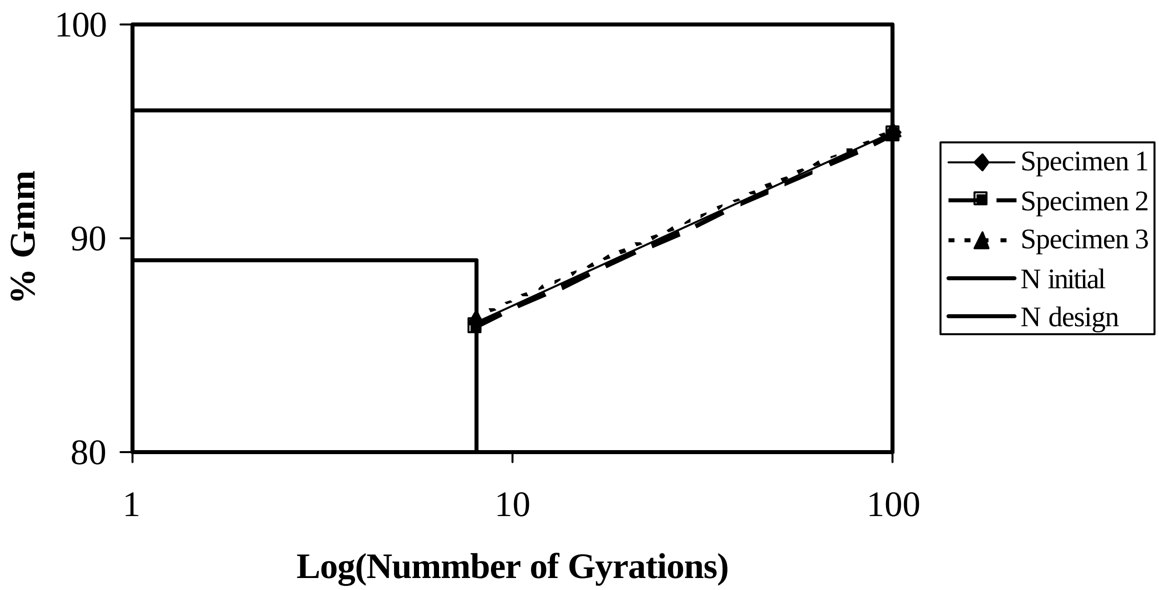


Table: Trial Blend No. 1 Compaction Results

Specimen	$\%G_{mm\ ini}$ $N_{ini}=8$	$\%G_{mm\ des}$ $N_{des}=100$	$\%G_{mm\ max}$ $N_{max}=160$
1	86.1 %	95.0 %	
2	85.8 %	94.9 %	n/a
3	86.2 %	95.1 %	

Figure: Gyratory Compaction Data



Q. For a design target of 4.0 % voids in total mix at N_{des} , is the asphalt binder content high or low?

Big Q. Should asphalt binder content be the main criteria for mixture design, Y or N?

Rationale for Compaction Criteria:

N_{ini} - *"Tenderness Check"* N_{ini} represents the mix during construction. Mixes that compact too quickly in the gyratory may have tenderness problems during construction.

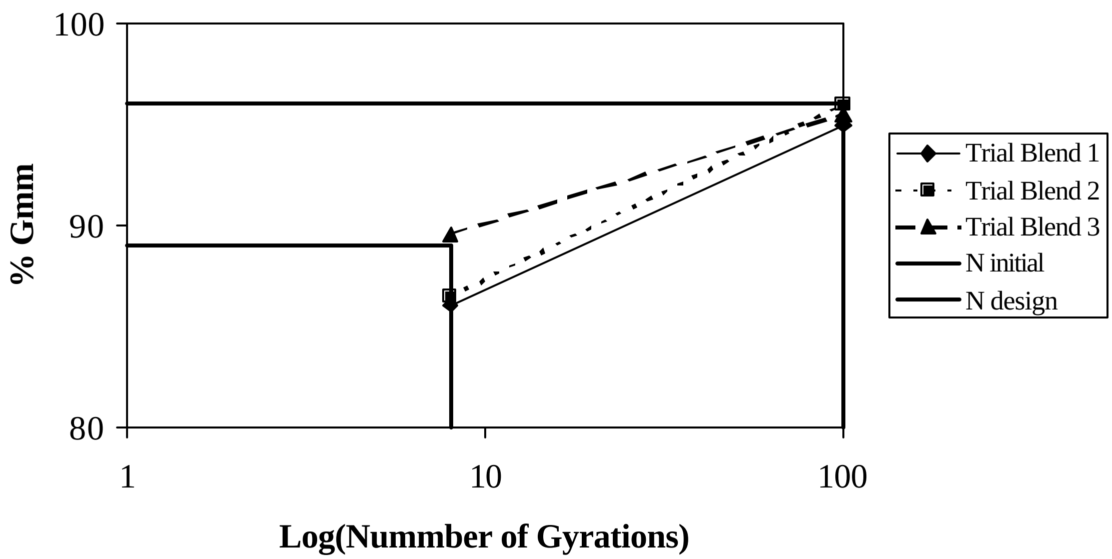
N_{design} - *"Volumetric Check"* N_{design} represents the mix after construction and trafficking. Mix volumetrics, (V_a , VMA, VFA), are compared to empirically based criteria.

N_{max} - *"Rutting Check"* Mixes that commonly rut have been compacted below 2 % voids under traffic. Mixes that compact below 2 % voids in the gyratory may have rutting problems. *(Applied only at the end of design procedure.)*

All three trial blends are compacted and the volumetric properties are determined. It is important to recognize that the trial blends are compacted at an estimated asphalt binder content. Under Superpave the design *(optimum)* asphalt binder content provides a mixture with four percent (4.0 %) voids in total mix (VTM or V_a) at the design number of gyrations (N_{design}); in addition to satisfying all other criteria. Only one of the trial blends at N_{design} yielded four percent (4.0 %) V_a . All this means is that the estimated trial asphalt contents were not exact. This will almost always be the case.

Table: Summary Superpave Gyratory Compaction Results

Trial Blend	Trial % AC	$\%G_{mm\ ini}$ $N_{ini} = 8$	$\%G_{mm\ des}$ $N_{des} = 100$	V_a $N_{des} = 100$	VMA $N_{des} = 100$
1	5.0	86.1	95.0	5.0	14.0
2	5.0	86.5	96.0	4.0	13.0
3	5.0	89.6	95.5	4.5	13.5



Superpave Gyratory Compaction Curves - Trial Blends

Figure: Trial Blend Gyratory Compaction Curves

Superpave provides a procedure for adjusting the volumetric results to reflect a four percent (4.0 %) void content at N_{des} . Upon completing the adjustments, the trial blends are then analyzed based on the established criteria.

Continuing: Estimate Trial Blends' Properties at 4.0% Air Voids (V_a)

The aggregate gradation governs the slope of the gyratory compaction curve (rate of compaction). In looking at the above compaction curves, it can be seen that the three trial blends produce different compaction rates. Because of this relationship, the blends' properties can be estimated.

1) Estimated asphalt binder content at 4.0 % V_a at N_{des}, $P_{b,est}$

$$P_{b,est} = P_{bt} - (0.4 * (4 - V_a \text{ at } N_{des}))$$

where:

P_{bt} = Trial percent asphalt binder content

V_a = Percent air voids in total mix at N_{des}

Trial Blend No. 1; $P_{b,est} = 5.0 - (0.4 * (4 - 5.0)) = 5.40$ %

2) Estimated voids in mineral aggregate (VMA) at N_{des}, at 4.0 % V_a, VMA_{est}

$$VMA_{est} = VMA \text{ at } N_{des} + C * (4 - V_a \text{ at } N_{des})$$

where:

C = constant (either 0.1 or 0.2)

C = 0.1 when V_a is less than 4.0%

C = 0.2 when V_a is 4.0% or greater

Trial Blend No. 1; $VMA_{est} = 14.0 + 0.2 * (4 - 5.0) = 13.8$ %

3) Estimated voids filled with asphalt (VFA) at N_{des}, at 4.0 % V_a, VFA_{est}

$$VFA_{est} = 100 * (VMA_{est} - V_a \text{ at } N_{des}) / VMA_{est}$$

Trial Blend No. 1; $VFA_{est} = 100 * (13.8 - 4) / 13.8 = 71.0$ %

4) Estimated Percent of Rice at N_{ini}, Est $\%G_{mm\ ini}$

$$\text{Est } \%G_{mm\ est\text{-}I} = \%G_{mm\ ini} - (4.0 - V_a \text{ at } N_{des})$$

Trial Blend No. 1; Est $\%G_{mm\ ini} = 86.1 - (4.0 - 5.0) = 87.1$ %

5) Estimated Percent of Rice at N_{max} , Est $\%G_{mm\ max}$

Est $\%G_{mm\ max} = \%G_{mm\ max} - (4.0 - V_a \text{ at } N_{des})$

Trial Blend No. 1; Est $\%G_{mm\ max} = 96.1 - (4.0 - 5.0) = 97.1\ \%$

6) Estimated Fines to Effective Asphalt Ratio, $F / P_{be,est}$

$$P_{be,est} = -(P_s * G_b) * \frac{(G_{se} - G_{sb})}{(G_{se} * G_{sb})} + P_{b,est}$$

$$F / P_{be,est} = \frac{\%P_{.075mm}}{P_{be,est}}$$

$$\textit{Trial Blend No. 1; } P_{be,est} = -(95.0 * 1.030) * \frac{(2.637 - 2.566)}{(2.637 * 2.566)} + 5.40 = 4.37\ \%$$

$$\textit{Trial Blend No. 1; } F / P_{be,est} = \frac{3.6\ \%}{4.37\ \%} = 0.82$$

Note: *The F / P_{be} ratio under Superpave is based on the effective asphalt binder content, not the total. This is sometimes referred to as the dust proportion.*

Table: Summary of Estimated Properties at 4 % V_a

Trial Blend	Estimated Properties				
	Binder P_b	**VMA at N_{des}**	**VFA at N_{des}**	**F / P_{be} Ratio**	**$\%G_{mm\ ini}$**
No. 1	5.4 %	13.8 %	71 %	0.82	87.1 %
No. 2	5.0 %	13.0 %	69 %	0.81	86.5 %
No. 3	5.2 %	13.4 %	70 %	1.15	90.1 %

Criteria	n/a	13.0 % minimum	65 - 75 % range	0.6 - 1.2 range	89.0 % maximum

Selection of the Design Aggregate Structure

Selecting the Design aggregate structure is the most difficult step. The estimated trial blends' properties are evaluated against the criteria:

° Trial Blend No. 1 passes ALL requirements.

° Trial Blend No. 2 *"passes"* ALL requirements.

However: VMA just meets the minimum requirement and the $\%G_{mm\ max}$ is just under the maximum criterion - during production it may be difficult to stay within the compaction criteria.

° Trial Blend No. 3 fails to meet all requirements.

$\%G_{mm\ ini}$ = 90.1 % > maximum criterion of 89 %

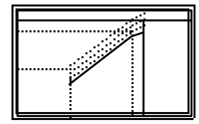
Select
Trial Blend
Number 1!

Select Design Aggregate Structure

Q. What if all three initial Trial Blends meet the design requirements. How would the Design Aggregate Structure be selected?

A. $ Economics $

③ SELECTION OF THE DESIGN ASPHALT BINDER CONTENT



Once the design aggregate structure is selected, Trial Blend No. 1 in this case, specimens are compacted at varying asphalt binder contents. The mixture properties are then evaluated to determine a design asphalt binder content. Superpave requires a minimum of two specimens compacted at each of the following asphalt contents, *(FHWA recommends three specimens compacted at each asphalt binder content)*:

- estimated asphalt binder content,
- estimated asphalt binder content ± 0.5 %, and
- estimated asphalt binder content + 1.0 %.

For Trial Blend No. 1, the asphalt binder contents for the mix design are 4.9%, 5.4%, 5.9%, and 6.4%. Two specimens are also prepared at each asphalt binder content for determination of maximum theoretical specific gravities (G_{mm}).

Table: Required Tests

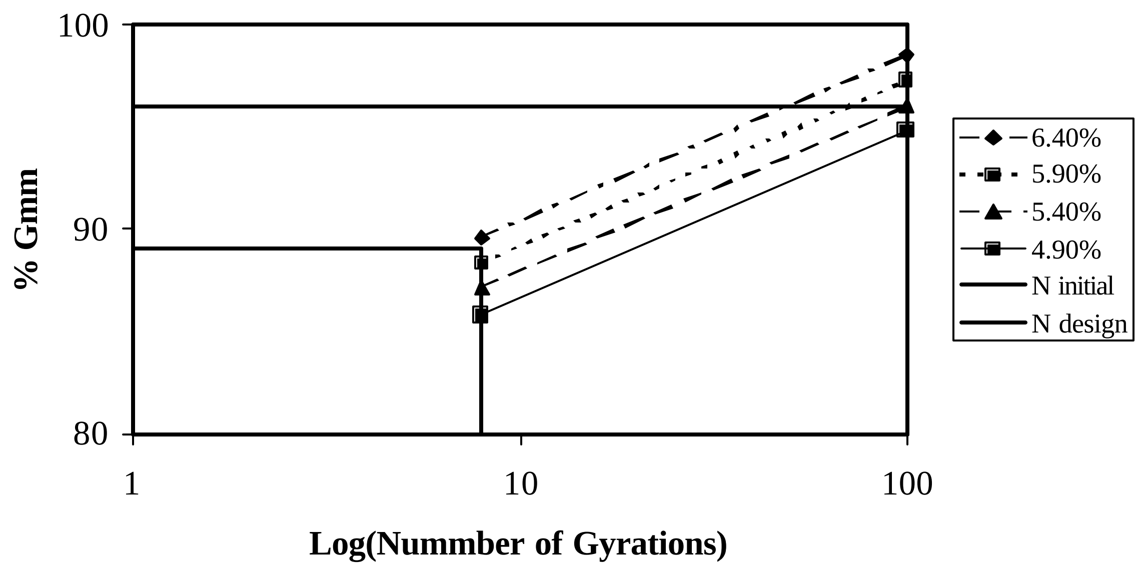
Batch Asphalt Binder Content	**Superpave Gyratory Compactor**	**Rice, G_{mm} (T 209)**
4.9%(-½%)	3 Specimens (4800 g/ea)	2 Tests (2000 g/ea)
5.4% Target*	3 Specimens	2 Tests
5.9%(+½%)	3 Specimens	2 Tests
6.4%(+1%)	3 Specimens	2 Tests
Total (73,600 g)	57,600 g	16,000 g

Lead States

Based upon the recommendations of NCHRP 9-9,
compact the design aggregate blend only to N_{des} .

Author's Note

Based on the calculations, 5.4 % is the estimated optimum asphalt content and should be the target



asphalt content for binder content selection. From a practical standpoint the results from the design aggregate structure selection can used to reduce the batching of all four binder contents.

Superpave Gyratory Compaction Curves

Figure: Design Gyratory Compaction Curves

Note: Each compaction curve represents the average of three compacted specimens.

Table: Compaction Test Results

Asphalt Content	**%$G_{mm\ ini}$**	**%$G_{mm\ des}$**
4.9 %	85.8 %	94.8 %
5.4 %	87.1 %	96.0 %
5.9 %	88.3 %	97.3 %
6.4 %	89.6 %	98.5 %
Criteria	≤ 89.0 %	= 96.0 %

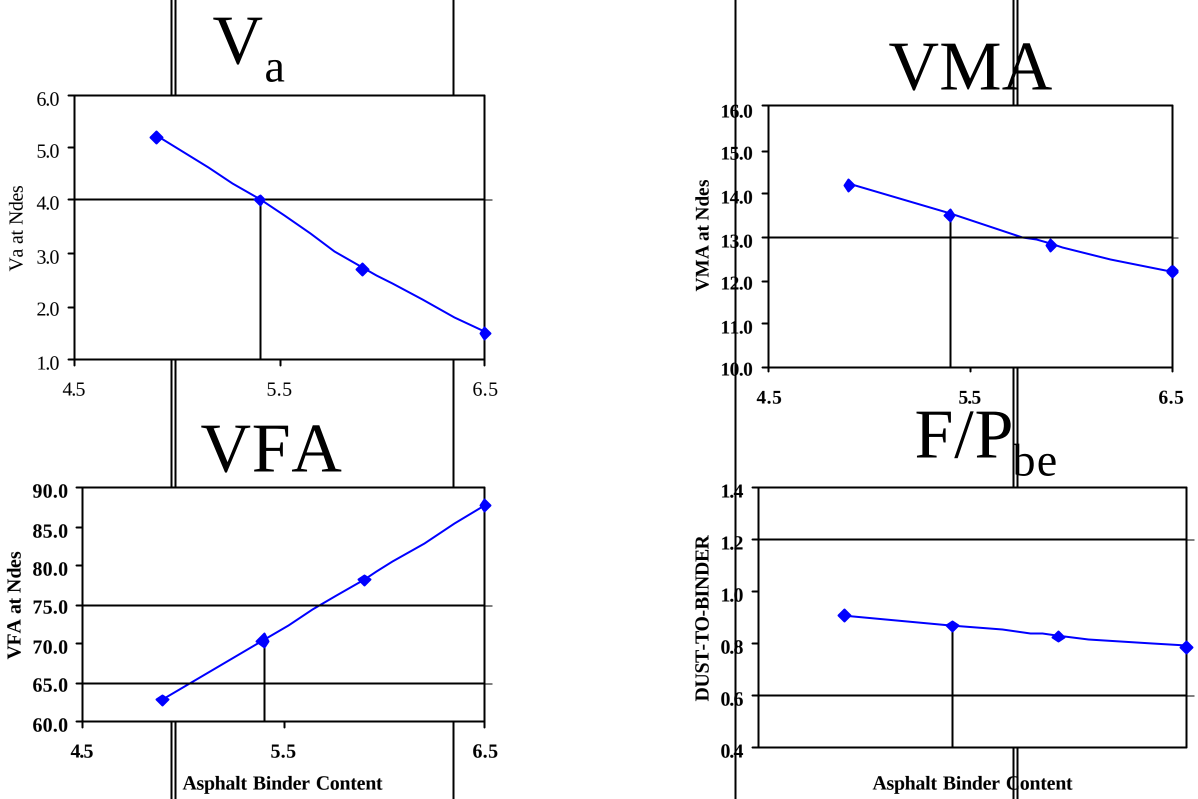
Table: Volumetric Test Results at N_{design}

Asphalt Content	V_a	VMA	VFA
4.9 %	5.2 %	14.2 %	62.7 %
5.4 %	4.0 %	13.5 %	70.4 %
5.9 %	2.7 %	12.8 %	78.1 %
6.4 %	1.5 %	12.2 %	87.7 %
Criteria	4.0 %	≥ 13.0%	65-75

Similar to the Marshall mix design procedure, the volumetric properties are plotted versus asphalt content. This provides a graphical means of determining the design asphalt binder content (see Figures). The design asphalt binder content is established as 4.0 % air voids (V_a) at N_{des} of 100 gyrations. In this simulation, the design asphalt binder content is 5.4 %. All other mixture properties are checked at the design asphalt binder content to verify that they meet the criteria. The design values for the 19.0 mm nominal mixture (Trial Blend No. 1) are indicated below:

Table: Summary of Design Mixture Properties at 5.4 % AC (P_b)

Mix Property	Results	Criteria



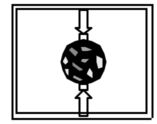
V_a at N_{des}	4.0 %	4.0 %
VMA at N_{des}	13.5 %	13.0 % Min
VFA at N_{des}	70 %	65 - 75 %
F / P_{be} Ratio	0.87	0.6 - 1.2
$\%G_{mm\ ini}$	86.9 %	$\leq$ 89 %
$\%G_{mm\ max}$	n/a	$\leq$ 98 %

Figures: Design Curves

The design aggregate structure at the optimum asphalt content is now checked at the maximum number of gyrations (N_{max}). As stated above, the compacted mixture should retain a minimum of 2 percent air voids, (maximum of 98 % G_{mm}), at N_{max}. For this project the following is determined:

$$\% \, G_{mm \, max} = 97.3 \, \% \leq 98 \, \% \text{ Criterion, (Okay)}$$

If the mix failed to meet the criterion, this indicates that a pavement made of this mix may be susceptible to rutting. The aggregate gradation should be adjusted accordingly. Different stockpile material may be required.



④ EVALUATION OF MOISTURE SENSITIVITY AASHTO T-283

The final step in the volumetric mix design process is to evaluate the moisture sensitivity of the design mixture. This step is accomplished by performing AASHTO T 283 on the design aggregate blend at the design asphalt binder content. Specimens are compacted to approximately 7.0% (±1.0%) air voids. One subset of three specimens is considered the control/unconditioned subset. The other subset of three specimens is the conditioned subset. The conditioned subset is subjected to partial vacuum saturation followed by an optional freeze cycle, followed by a 24 hour heating cycle at 60°C. All specimens are tested to determine their indirect tensile strengths. The moisture sensitivity is determined as a ratio of the tensile strengths of the conditioned subset divided by the tensile strengths of the control subset. The table below indicates the moisture sensitivity data for the mixture at the design asphalt binder content.

Table: AASHTO T 283 Results

Samples	**Superpave Gyratory Compactor**	**Indirect Tensile Strength**
Un-conditioned Specimens (Dry)	3 Specimens compacted to 7 % V_a (14,400 g)	872 k Pa
Conditioned Specimens (Wet)	3 Specimens compacted to 7 % V_a (14,400 g)	721 k Pa
	% TSR	82.7 % (Ok)
	Superpave Criteria	80.0 % Min

The minimum criteria for tensile strength ratio is 80.0 %. The design blend (82.7 %) meets the minimum requirement. At this point, Superpave Volumetric Mixture Design is complete.

Author's Note

The criteria for TRS is based on experience gained from analysis 4 inch Marshall specimens. Research conducted under NCHRP evaluated 150 mm (6 inch) SGC specimens in the AASHTO T -283 procedure. The conditioning procedure was not as severe with the larger specimens. Most designers are using T-283 with SGC specimens. However, others are using 4 inch specimens, while others are using the SGC specimens with 100% saturation, while still others are using other test procedures. The use of AASHTO T-283 is not required to design a Superpave mix. However, some method of moisture sensitivity should be employed.

APPENDICES

MIXING AND COMPACTION TEMPERATURE DETERMINATION

SGC Evaluation Specification, on CD

RAP Guidelines, on CD

MIXING AND COMPACTION TEMPERATURE DETERMINATION

Alternate Method using Mathematics

Variables:

μ = viscosity, in centiStokes

$u = Log_{10}(Log_{10}(\mu))$

T = temperature, in Kelvin

$t = Log_{10}(T)$

m = slope of the line

b = Y axis intercept (Log-Log(Viscosity))

Data: μ_1 = 379 centiStokes, viscosity at first test temperature T_1 = 135° + 273° = 408°K

$u_1 = Log_{10}(Log_{10}(\mu_1)) = 0.4114$

$t_1 = Log_{10}(T_1) = 2.611$

μ_2 = 106 centiStokes, viscosity at second test temperature T_2 = 160° + 273° = 433°K

$u_2 = Log_{10}(Log_{10}(\mu_2)) = 0.3065$

$t_2 = Log_{10}(T_2) = 2.637$

$m = (u_2 - u_1)/(t_2 - t_1) = -4.035$

$b = u_1 - m * t_1 = 10.9468$

Calculations:

$t_x = (u_x - b)/m$,

where $u_x = Log_{10}(Log_{10}(150, 190, 250, \& 310)) = 0.3377, 0.3577, 0.3798, \& 0.3964$

$T_x = 10^{tx}$

Such that, $T_x = 10^{(ux - 10.9468)/(-4.035)}$

Mixing: $10^{(0.3377 - 10.9468)/(-4.035)}$ = 425.9°K

425.9°K (153°C) to

421.0°K (148°C)

Compaction: 415.7°K (143°C) to 411.8°K (139°C)