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Asphalt paving mix design — Code of practice

EAST AFRICAN COMMUNITY

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Foreword

Development of the East African Standards has been necessitated by the need for harmonizing requirements governing quality of products and services in the East African Community. It is envisaged that through harmonized standardization, trade barriers that are encountered when goods and services are exchanged within the Community will be removed.

The Community has established an East African Standards Committee (EASC) mandated to develop and issue East African Standards (EAS). The Committee is composed of representatives of the National Standards Bodies in Partner States, together with the representatives from the public and private sector organizations in the community.

East African Standards are developed through Technical Committees that are representative of key stakeholders including government, academia, consumer groups, private sector and other interested parties. Draft East African Standards are circulated to stakeholders through the National Standards Bodies in the Partner States. The comments received are discussed and incorporated before finalization of standards, in accordance with the Principles and procedures for development of East African Standards.

East African Standards are subject to review, to keep pace with technological advances. Users of the East African Standards are therefore expected to ensure that they always have the latest versions of the standards they are implementing.

The committee responsible for this document is Technical Committee EASC/TC 028, *Construction of Roads, Rail, Air and Water Transport Infrastructure.*

Attention is drawn to the possibility that some of the elements of this document may be subject of patent rights. EAC shall not be held responsible for identifying any or all such patent rights.

Introduction

Roads are vital to economic development, but can be very expensive, especially if the performance of the road's surface is poor. It is therefore important that suitable methods of design are developed for the wide range of conditions that road surfaces are expected to endure. The principal roads in most countries are surfaced with Hot Mix Asphalt (HMA), that is, a mixture of aggregate materials bound together with bitumen. Asphalt pavements are the predominant pavement type in the world. Asphalt is used for all types of applications, from residential streets to expressways, from parking lots to harbour facilities, and from bike paths to airport runways. Depending on traffic, climate, available materials and the location within the pavement structure, the type of mix selected and mix design criteria will be different. Road conditions are, however, not static; for example, continuing developments in vehicle and tyre designs often increase the stresses that are applied to the road. In most countries, traffic levels are also increasing, sometimes beyond the limits of the empirical data on which designs are based. In some countries there is a shortage of materials of sufficient quality for road surfaces and therefore innovative solutions need to be sought. Environmental concerns are becoming increasingly important and influence the techniques available; for example, encouraging the recycling of existing materials. For these reasons, amongst others, research into improving the design and performance of HMA road surfaces continues to be undertaken.

This standard was prepared with these goals in mind. It contains the latest information for the design of asphalt paving mixtures to meet the demands of modern traffic conditions and to ensure optimal performance of asphalt pavements. The overall objective for the design of asphalt paving mixes is selecting the gradation of aggregates and proportioning of asphalt that yields a mix having the desirable properties.

Successful mix design requires understanding the basic theory behind the steps and following the intent of the written instructions. It also includes having proper training in laboratory techniques and effectively interpreting the results of laboratory tests. A properly designed asphalt mixture provides a balance of engineering properties and economics that ensures a durable pavement that satisfies both its users and owners. The overall objective of this standard guideline is to help engineers responsible for roads and gives guidance on the design, manufacture and construction of HMA pavement materials in the East African Community region.

Asphalt paving mix design — Code of practice

1 Scope

This Draft East African Standard provides guidelines on the design of asphalt paving mixtures to be used for general asphalt pavements.

These guidelines are applicable to the design of Hot Mix Asphalts (HMA) produced from new (virgin) asphalt materials and Reclaimed (Recycled) Asphalt Materials in the design process.

NOTE The guidelines for mix design given in Clause 7 are based on commonly used asphalt design methods mainly Superpave HMA Mix Design System, Marshall Method of asphalt mix design and Hveem method of asphalt mix design.

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

D2041/D2041M-19, *Standard Test Method for Theoretical Maximum Specific Gravity and Density of Asphalt Mixtures*

ASTM C127, *Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate*

ASTM C128 *Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Fine Aggregate*

ASTM D6926, *Preparation of Bituminous Mixtures Using Marshall Apparatus*

ASTM D6927, *Standard Test Method for Marshall Stability and Flow of Bituminous Mixtures*

D1188/D1188M-22, *Standard Test Method for Bulk Specific Gravity and Density of Compacted Asphalt Mixtures Using Coated Samples*

D2726/D2726M-21, *Standard Test Method for Bulk Specific Gravity and Density of Non-Absorptive Compacted Asphalt Mixtures*

D6927-22, *Standard Test Method for Marshall Stability and Flow of Asphalt Mixtures*

D5581-07A (2021), *Standard Test Method for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus (152.4mm/6 in. Diameter Specimen)*

D1560-15, *Standard Test Methods for Resistance to Deformation and Cohesion of Asphalt Mixtures by Means of Hveem Apparatus*

D2041/D2041M-19, *Standard Test Method for Theoretical Maximum Specific Gravity and Density of Asphalt Mixtures*

D5404/D5404M-21, *Standard Practice for Recovery of Asphalt Binder from Solution Using the Rotary Evaporator*

D6752/D6752M-23, *Standard Test Method for Bulk Specific Gravity and Density of Compacted Asphalt Mixtures Using Automatic Vacuum Sealing Method*

M 323, *Standard Specification for Superpave Volumetric Mix Design,*

R 30, *Standard Practice for Laboratory Conditioning of Asphalt Mixtures,*

T 30, *Standard Method of Test for Mechanical Analysis of Extracted Aggregate*

T 164, *Standard Method of Test for Quantitative Extraction of Asphalt Binder from Hot Mix Asphalt (HMA),*

T 308, *Standard Method of Test for Determining the Asphalt Binder Content of Asphalt Mixtures by the Ignition Method*

T 209, *Standard Method of Test for Theoretical Maximum Specific Gravity (Gmm) and Density of Asphalt Mixtures*

M 323, *Standard Specification for Superpave Volumetric Mix Design,*

M 320, *Standard Specification for Performance-Graded Asphalt Binder*

3 Terms and definitions

For the purposes of this document, the following terms and definitions apply.

ISO and IEC maintain terminological databases for use in standardization at the following addresses:

— IEC Electropedia: available at http://www.electropedia.org/

— ISO Online browsing platform: available at http://www.iso.org/obp

3.1
asphalt
a mixture of asphalt binder (bitumen) and aggregates. The mixture may also include other materials.

3.2
asphalt pavement
a structure consisting of one or more prepared layers of asphalt mix atop one or more supporting layers of unbound, modified, or treated subgrade, subbase or base materials

3.3
asphalt mix
homogeneous paving product consisting of sized mineral aggregates, possibly including additives and filler, collectively and uniformly coated with binder

3.4
hot mix
asphalt mix in which the aggregates and asphalt binder(bitumen) are heated before mixing

3.5
Bitumen/asphalt binder
dark brown to black cement-like residuum obtained from the distillation of suitable crude oils, naturally occurring sources, or combinations thereof.

3.6
asphalt mix stability
maximum load resistance in Newton, that the standard test specimen will develop at 60°C when tested as outlined in methodologies described in Clause 6.

3.7
asphalt mix flow value
total deformation, in units of 0.25 mm, occurring in the specimen between no load and the point of maximum load during the stability test.

3.8 Terms related to asphalt mix classification

3.8.1
Dense-graded asphalt mixes
asphalt mix with a well-distributed aggregate gradation throughout the entire range of sieves used.

Note 1 to entry: It is the most commonly specified type of mix and can be used in the base, intermediate layers and surface of a pavement structure.

3.8.2
open-graded asphalt mixes
asphalt mixture designed with a large volume of air voids (typically 18 to 22 percent) so that water will readily drain through the pavement layer.

Note 1 to entry: It is used as an Open-Graded Friction Course (OGFC) to provide a skid-resistant pavement surface and as a porous base layer (also called Asphalt Treated Permeable Base, or ATPB) to provide for positive drainage under either an asphalt or Portland cement concrete pavement surface.

3.8.3
gap-graded or Stone Matrix Asphalt (SMA)
an asphalt mixture with high-coarse aggregate content (typically 70 to80 percent), a high asphalt content (typically more than 6 percent) and a high-filler content (approximately 10 percent by weight).

Note 1 to entry: Gap-graded or Stone Matrix Asphalt (SMA) is durable mixture that has excellent stone-on-stone contact and that is very resistant to rutting.

3.8.4
Warm Mix Asphalt (WMA)
any mix produced at lower temperatures using a variety of technologies while maintaining the workability required to be successfully placed.

3.9 Terms related to mix specific gravity

3.9.1
binder Specific Gravity G_b
ratio of the mass of a unit volume of binder to the mass of the same volume of water. Binder specific gravity typically ranges from 1.00 to 1.05.

3.9.2
bulk (dry) Specific Gravity G_{sb}
ratio of the oven-dry mass of a unit volume of aggregate (including both the impermeable and water-permeable void volumes) to the mass of the same volume of water.

3.9.3
apparent Specific Gravity G_{sa}
the ratio of the oven-dry mass of a unit volume of aggregate (including only the impermeable void volumes) to the mass of the same volume of water.

3.9.4
effective Specific Gravity G_{se}
ratio of the oven-dry mass of a unit volume of aggregate (including both the impermeable void volumes and the water permeable voids not filled with absorbed asphalt) to the mass of the same volume of water.

3.9.5
theoretical Maximum Specific Gravity G_{mm}
the ratio of the oven-dry mass of a unit volume of asphalt mixture (including the volumes of the aggregate and binder only) to the mass of the same volume of water.

3.9.6
bulk Specific Gravity G_{mb}
the ratio of the oven-dry mass of a unit volume of asphalt mixture (including the volumes of aggregate, binder and air) to the mass of the same volume of water. G_{mb} is applicable to any laboratory- or field-compacted specimen including cores, beams, slabs, etc.

3.10 Terms related to mixture volumetric parameters

3.10.1
percent Air Voids P_a
The volume of air voids in a compacted mixture, expressed as a percentage of the total mix volume.

3.10.2
voids in the Mineral Aggregate (VMA)
voids created by the aggregate structure of a compacted asphalt mixture, expressed as a percentage of the total mix volume. VMA represents the volume of air voids and effective (nonabsorbed) asphalt binder.

3.10.3
voids Filled with Asphalt (VFA)
percentage of the VMA filled with effective (nonabsorbed) asphalt binder.

3.10.4
percent Aggregate P_s
total percentage of aggregate in the asphalt mixture, expressed as a percentage of the total mix mass.

3.10.5
percent Binder P_b
total percentage of asphalt binder in the asphalt mixture, expressed as a percentage of the total mix mass. Note that $P_s + P_b = 100\%$.

3.10.6
percent Binder Effective P_{be}
functional portion of the asphalt binder that coats the aggregate in the asphalt mixture but is not absorbed into the aggregate, expressed as a percentage of the total mix mass.

3.10.7
percent Binder Absorbed P_{ba}
portion of the asphalt binder that is absorbed into the aggregate, expressed as a percentage of the total aggregate mass.

3.10.8
specific gravity
dimensionless number defined as the ratio of the density of a material to the density of water (assumed to be 1.000 g/cm3 at temperatures used in asphalt testing).

3.11 Terms related to desired properties considered for mix design

3.11.1
stability
resistance to permanent deformation results from the accumulation of small amounts of unrecoverable strain (small deformations) from repeated loads applied to the pavement.

Note 1 to entry: Stability depends primarily on the internal friction provided by the aggregate particles and to a lesser extent the cohesion provided by the asphalt binder. Unstable pavement develops ruts and other signs of mixture shifting.

3.11.2
fatigue resistance
resistance to repeated bending under wheel loads (traffic). The result of a fatigue failure is fatigue cracking, often called alligator cracking.

3.11.3
moisture resistance—impermeability
ability of asphalt mix to prevent water or water vapor from penetrating between the asphalt film and the aggregates, thereby preventing the breaking of adhesive bond between the aggregate and the asphalt binder film.

3.11.4
durability
ability to resist factors such as aging of the asphalt, disintegration of the aggregate and stripping of the asphalt film from the aggregate.

3.11.5
skid resistance
ability of an asphalt surface to minimize skidding or slipping of vehicle tires, particularly when the roadway surface is wet.

3.11.6
workability
the ease with which a paving mixture can be placed and compacted.

4 Symbols and/or abbreviated terms

HMA Hot Mix Asphalt

RAP Reclaimed Asphalt Pavement

OGFC Open-Graded Friction Course

ATPB Asphalt Treated Permeable Base

SMA Stone Matrix Asphalt

WMA Warm Mix Asphalt

DAS Design Aggregate Structure

CKE Centrifuge Kerosene Equivalent

VFA Voids Filled with Asphalt

5 Volumetric Properties of Compacted Paving Mixtures

5.1 General

The volumetric properties of a compacted paving mixture are important criteria by which the quality of an asphalt mixture is evaluated. The volumetric properties are determined using the mass and/ or volume measurements of a mixture and its constituent components (binder, aggregate, air). The term “volumetric,” as applied in the asphalt industry, actually uses measurements of an asphalt mixture by both mass (M) and volume (V) to determine various percentages (P). Volumetric properties are often specified design elements of the total mix, the aggregate only or the binder only. The relationship between mass and volume is determined by the material’s specific gravity (G).

NOTE The principal of using specific gravity is essential in that actual laboratory measurements are by mass, yet mixture criteria are a function of volume. Usage of specific gravity principles enables us to readily calculate mass or volumetric data based on the available data. Some specific gravity values use an aggregate volume that also includes the water-permeable voids, while others include only the portion of the water-permeable voids not filled with absorbed asphalt. (Because liquid water is always at a lower viscosity than asphalt binder, the asphalt can never penetrate the aggregate voids as much as water.)

It is helpful to represent a compacted mixture in terms of the different masses and volumes used in volumetric calculations. This representation, shown in Figure 1, is called a phase diagram. It breaks down the components into air, effective asphalt (nonabsorbed), absorbed asphalt and aggregate.

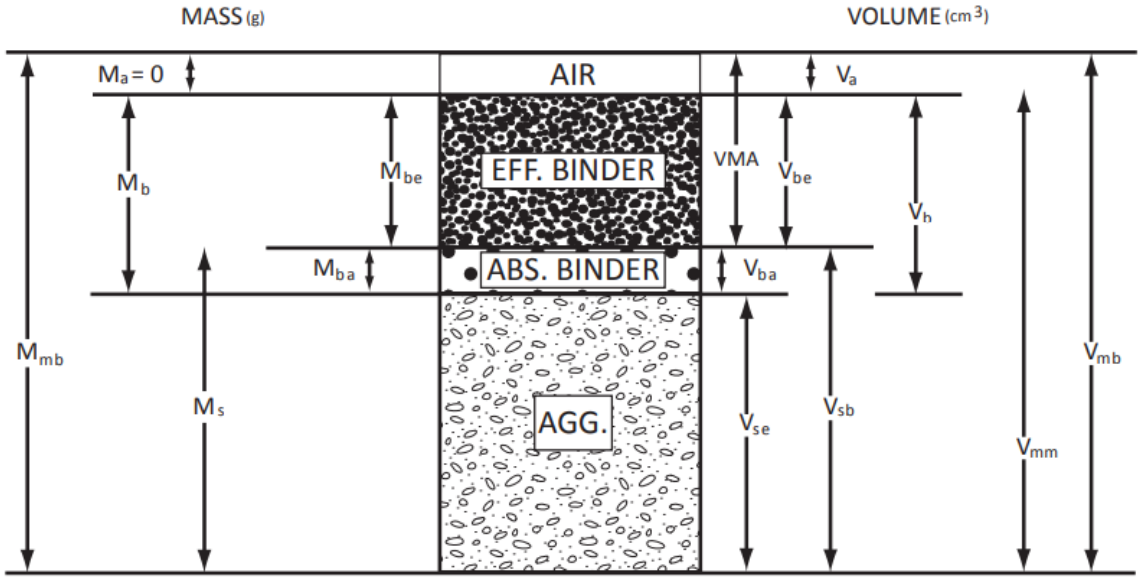

Figure 1 — Phase Diagram

The various specific gravity types discussed in this clause are either directly determined or calculated from laboratory tests on the binder, aggregates or asphalt mixture. Several equations have been derived to allow calculation of each volumetric property without the use of a phase diagram. The equations shown in this clause are simplified, as the value of density of water (ρ) is assumed to be 1.000.

5.2 Bulk (dry) specific gravity of aggregate

It is recommended that the bulk (dry) specific gravity (G_{sb}) of each aggregate be determined on samples submitted for mix design. Some stockpiles will be essentially coarse (retained on the No. 4[4.75 mm] sieve), some will be fine (passing the No.4 [4.75 mm] sieve) and some will have both coarse and fine portions.

5.2.1 Determining bulk (dry) specific gravity of coarse aggregate

The G_{sb} of coarse aggregate is determined using ASTM C127. The size of the test sample is specified and determined by the nominal maximum aggregate size. This procedure requires that the dry aggregate be saturated to determine the volume of the aggregate plus the water-permeable voids.

$$G_{sb} = \frac{m}{v\rho} = \frac{mass\ of\ oven\text{-}dry\ aggregate}{(volume\ of\ aggregate + water\text{-}permeable\ voids) \times \rho}$$

Notice that this equation mirrors the equation in the test procedure:

$$G_{sb} = \frac{A}{B - C}$$

where

Gsb is bulk (dry) specific gravity of the aggregate

A is mass of the oven-dry test sample

B is mass of the saturated surface-dry (SSD) test sample in air

C is mass of the saturated sample in water (ρ is not shown because its numerical value is 1)

Therefore, B–C = volume of the aggregate plus the water-permeable voids.

5.2.2 Determining fine aggregate G_{sb}

The fine aggregate G_{sb} is determined using ASTM C128. The dry aggregate is again saturated to account for the volume of the aggregate plus the water-permeable voids.

$$G_{sb} = \frac{m}{v\rho} = \frac{mass\ of\ oven\text{-}dry\ aggregate}{(volume\ of\ aggregate + water\text{-}permeable\ voids) \times \rho} = \frac{A}{B + S - C}$$

where

Gsb is bulk (dry) specific gravity of the aggregate

A is mass of the oven-dry test sample

B is mass of the pycnometer filled with water

S is mass of the saturated surface-dry (SSD) specimen

C is mass of pycnometer with specimen and water to calibration mark

This time, $B + S - C$ = volume of the aggregate plus the water-permeable voids.

5.2.2.1 Determining mineral filler G_{sb}

The bulk specific gravity of mineral filler is difficult to determine accurately. However, the apparent specific gravity (G_{sa}) of mineral filler is more easily determined. This can be done for filler only, as the amount of mineral filler added is typically small and the difference between G_{sb} and G_{sa} is relatively small.

5.2.3 Determining the composite G_{sb} for one stockpile

For stockpiles that include both a coarse and fine fraction, one value must be determined for the stockpile. The average G_{sb} can be calculated as follows:

$$G_{sb} = \frac{P_{coarse} + P_{fine}}{\frac{P_{coarse}}{G_{coarse}} + \frac{P_{fine}}{G_{fine}}}$$

where

G_{sb} bulk (dry) specific gravity of the aggregate

P_{coarse} percentage by weight retained on the No. 4 (4.75 mm) sieve

P_{fine} percentage by weight passing the No. 4 (4.75 mm) sieve

G_{coarse} bulk (dry) specific gravity of the coarse fraction

G_{fine} bulk (dry) specific gravity of the fine fraction

5.2.4 Calculate the G_{sb} for the aggregate blend

Once the bulk (dry) specific gravity for each stockpile has been determined, the combined bulk (dry) specific gravity for the total aggregate blend is calculated as follows:

$$G_{sb} = \frac{P_1 + P_2 + \ldots + P_n}{\frac{P_1}{G_1} + \frac{P_2}{G_2} + \ldots + \frac{P_n}{Gn}}$$

where

Gsb is bulk (dry) specific gravity of the aggregate

P_1, P_2, P_n is percentages by weight of aggregates 1, 2, through n

G_1, G_2, G_n is bulk (dry) specific gravity of aggregates 1, 2, through n

NOTE This equation is useful for estimating G_{sb} during trials in the design process. The calculated coarse and fine G_{sb} can be verified by batching the combined aggregates, splitting them on the 4.75-mm sieve and determining the coarse and fine G_{sb} for the design. This process of splitting the aggregate blend on the 4.75- mm sieve and only running G_{sb} values on the coarse and fine fractions of the blend is often utilized for mix design verification and quality control testing on plant-produced mix in the field.

5.2.5 Calculate the G_{sa} and absorption for the aggregate blend

Laboratory testing to determine the bulk specific gravity (G_{sb}) also provides data to easily determine two additional aggregate properties, the apparent specific gravity (G_{sa}) and the water absorption of the aggregate. These calculations are not required to determine mixture volumetric properties; however, they are valuable

tools for the mix designer to monitor. The absorption of the aggregate indicates several characteristics of the final mixture. Highly absorptive aggregates will require additional binder to fill the permeable voids in the aggregate, which increases cost. It is not uncommon for aggregates to absorb a binder amount equal to 40–80 percent of the water-permeable voids.

In order to determine these values for the total blend of aggregate, the methodology used will depend on the manner in which the G_{sb} was determined. If G_{sb} testing was conducted on individual stockpiles, then the G_{sa} and absorption will need to be determined for each stockpile and then combined to determine the final values for the blend. If individual G_{sb} samples were determined for the coarse and fine fractions of any individual stockpile, then the equation shown in clause 5.2.1 can be used to determine the G_{sa} and absorption values for the stockpile.

Once G_{sa} values of individual stockpiles are known, they will need to be combined into one value for the blend. This can be accomplished using the same equation shown in section 5.2.4, but for G_{sa} instead of G_{sb}. The equation for calculating the absorption of the blend is shown in section 5.2.5.2. If G_{sb} data are determined directly from the blend (during a mixture verification or a field sample obtained from the belt carrying aggregate into the plant), the designer can simply use the equation from section 6.2.3 to directly determine the G_{sb}, G_{sa} and absorption data for the blend.

5.2.5.1 Apparent specific gravity (G_{sa})

G_{sa} is the ratio of the mass of the oven-dry aggregate to the volume of the aggregate excluding the volume of the voids occupied by absorbed water. The G_{sa} volume is less than the volume used to calculate the G_{sb}; therefore, the G_{sa} value will always be larger than the G_{sb} value.

$$G_{sa} = \frac{m}{v \times \rho} = \frac{mass\ of\ oven\text{-}dry\ aggregate}{(bulk\ volume\ of\ aggregate - volume\ of\ water\text{-}permeable\ voids) \times \rho} = 2.700$$

Notice that this equation mirrors the equation in the test procedure:

$$G_{sa} = \frac{A}{A - C}$$

where

G_{sa} is apparent specific gravity of the aggregate

A is mass of the oven-dry test sample

C is mass of the saturated sample in water (ρ is not shown because its numerical value is 1)

Therefore, A − C = apparent volume of the aggregate minus the water-permeable voids.

5.2.5.2 Water absorption (A)

The amount of water absorption is determined from the G_{sb} test data. The absorptiveness of aggregate is of significant interest to the mixture designer and specifier. Absorption can be an indicator regarding aggregate quality along with increased binder demand. The binder absorption is typically 40–80 percent of the water absorption rate. The water absorption rate is calculated by the following equation.

$$Absorption, \% = \frac{(B - A)}{A} \times 100$$

where

B is mass of the saturated surface-dry sample

A is mass of the oven-dry test sample

The average water absorption for the total aggregate blend is calculated as follows:

$$A = \frac{(P_1)(A_1)+(P_2)(A_2)+\ldots+(P_n)(A_n)}{100}$$

where

P_1, P_2, P_n is percentages by weight of aggregates 1, 2, through n

A_1, A_2, A_n is absorption of aggregates 1, 2, through n

5.3 Calculating G_{mm} at trial binder contents

The theoretical maximum specific gravity of an asphalt mixture (G_{mm}), is the specific gravity of the binder coated aggregate only, with no air voids. In order to calculate the volumetric properties of a mixture, a G_{mb} and G_{mm} must be determined at each trial binder percentage utilized in the mix design. Compaction procedures provide the G_{mb} values for each sample, which are then averaged for each trial binder content. The appropriate G_{mm} value must also be determined at each trial binder percentage.

5.3.1 Determining the G_{se} — effective specific gravity of the aggregate

When only one G_{mm} is conducted in the laboratory, the designer should select a binder content that is equal to or greater than the anticipated design binder content in order to assure thorough coating and minimize water intrusion into the aggregate during vacuum testing. After finding the average of two G_{mm} samples at a single binder content (or if desired, at every trial binder percentage), G_{se} can be calculated using the following equation:

$$G_{se} = \frac{P_s}{\frac{100}{G_{mm}} - \frac{P_b}{G_b}}$$

where

G_{se} is effective specific gravity of aggregate

P_s is percentage of aggregate by total mix weight

P_b is percentage of binder by total mix weight, at which the G_{mm} test was performed

G_{mm} is maximum specific gravity of paving mixture

G_b is specific gravity of binder

5.3.2 Determining G_{mm} at other binder contents

The G_{se} used to calculate G_{mm} at each of the other binder contents. This step is not necessary if the designer has performed G_{mm} testing at each trial binder percentage. G_{mm} can be visually determined from the phase diagram in Figure 1 and is defined by the following relationship:

$$G_{mm} = \frac{M_{mb}}{V_{mm}\rho}$$

where

G_{mm} is maximum specific gravity of asphalt mixture

P_s is percentage of aggregate by total mix weight

P_b is percentage of binder by total mix weight

$P_s + P_b = 100$

G_{se} is effective specific gravity of aggregate

G_b is specific gravity of binder

M_{mb} is bulk mass of paving mixture (which would be the same as Mmm, since the air has no mass), typically in g

V_{mm} is volume of aggregate and binder, typically in cm3 ρ = density of water, 1.000 g/cm3

NOTE The binder content increases, G_{mm} always decreases. This is because the percentage of aggregate, which has a higher specific gravity, necessarily decreases for a unit volume with an increase in the percentage of binder, which has a lower specific gravity.

5.4 Percent air voids in compacted mixture, P_a

Pa is the percentage of air voids by volume and V_a as the measured volume of air voids. They consist of the small air spaces between coated aggregate particles. The property Pa can be visually determined from the phase diagram in Figure 1, and is defined by the following relationship:

$$P_a = 100 \times \frac{V_a}{V_{mb}}$$

Although the Pa can be calculated several different ways, the following equation is most commonly used:

$$P_a = 100 - \frac{100 \times G_{mb}}{G_{mm}}$$

where

P_a is air voids in compacted mixture, percentage of total volume

G_{mm} is maximum specific gravity of paving mixture

G_{mb} is bulk specific gravity of paving mixture

5.5 Percent VMA in compacted mixture

The VMA is calculated on the basis of the bulk specific gravity of the aggregate and is expressed as a percentage of the bulk volume of the compacted paving mixture. Therefore, the VMA can be calculated by subtracting the volume of the aggregate determined by its bulk specific gravity from the bulk volume of the compacted paving mixture. VMA can be visually determined from the phase diagram in Figure 1 and is defined by the following relationships:

$$VMA = 100 \times \frac{V_a + V_{be}}{V_{mb}}$$

VMA is most readily calculated utilizing the following equation:

$$VMA = 100 - \frac{G_{mb} P_s}{G_{sb}}$$

where

VMA is voids in the mineral aggregate

G_{mb} is bulk specific gravity of paving mixture

P_s is percentage of aggregate by total mix weight

G_{sb} is bulk (dry) specific gravity of the aggregate

V_a is volume of voids in compacted mixture, typically in cm^3

V_{be} is volume of the effective (nonabsorbed) binder, typically in cm^3

V_{mb} total volume of compacted mixture, typically in cm3

The equations shown above are for analyzing mixture compositions that are determined as percent by weight of the total mixture. If the mixture composition is determined as percent by weight of aggregate, the following equation must be utilized to calculate VMA:

$$VMA = 100 - \frac{G_{mb}}{G_{sb}} \times \frac{100}{100 + P_b} \times 100$$

NOTE Because the VMA does not include the water permeable voids in the aggregate, the bulk dry G_{sb} must be utilized in calculating VMA. Since the target air voids (Pa) typically remains the same, the VMA must increase to allow sufficient room for the additional asphalt binder.

5.6 Percent VFA in compacted mixture

VFA, like VMA, also tends to increase as the mix becomes finer and gains more total aggregate surface area. The VFA can be calculated with either of the following equations. VFA can be illustrated by the phase diagram in Figure 1 and can be calculated by the following equation :

$$VFA = 100 \times \frac{V_{be}}{V_{be} + V_a}$$

VFA is most readily calculated with the following equation:

$$VFA = 100 \times \frac{VMA - P_a}{VMA}$$

where

VFA is voids filled with asphalt

VMA is voids in the mineral aggregate

P_a is air voids in compacted mixture, percentage of total volume

V_{be} is volume of the effective (non- absorbed) binder, typically in cm^3

V_a is volume of voids in compacted mixture, typically in cm^3

5.7 Binder absorption

The percent binder absorption (P_{ba}) is the percentage by mass of binder that is absorbed into the aggregate. It is assumed that the amount of binder absorbed into the aggregate is a constant value; therefore, it is calculated based on the mass of the aggregate. Note that if the absorption was calculated based on the total mass of the mix, the percent absorption would change based on the amount of binder added to the mix.

P_{ba} can be visually determined from the phase diagram in Figure 1 and is defined by the following relationship:

$$P_{ba} = 100 \times \frac{M_{ba}}{M_s}$$

P_{ba} is most readily calculated with the following equation:

$$P_{ba} = 100 \times \frac{(G_{se} - G_{sb})}{(G_{se} \times G_{sb})} \times G_b$$

where

P_{ba} is absorbed binder, percentage by mass of aggregate

G_{se} is effective specific gravity of aggregate

G_{sb} is bulk (dry) specific gravity of the aggregate

G_b is specific gravity of binder

M_{ba} is mass of the absorbed asphalt, typically in grams

M_s is mass of the aggregate, typically in grams

5.8 Effective binder content of a paving mixture

The effective binder content (P_{be}) of a paving mixture is the percentage by mass of binder that stays on the outside of aggregate particles and is not absorbed. It is effective or usable, as the

"glue" that binds the mix together and governs the performance of an asphalt paving mixture. Note that P_{be} is expressed as a percentage of the total mix mass. That means that mathematically, $P_{ba} + P_{be} \neq P_b$, the total binder content, because P_{ba} is a percentage of the total aggregate and P_{be} is a percentage of the total mix. However, the mass of the total aggregate and the mass of the total mix are so close in magnitude that in a practical sense, when calculated to the nearest 0.1 percent, the absorbed and effective binder contents added together usually equals the total binder content. It can be calculated as follows:

From the phase diagram, P_{be} can be defined as:

$$P_{be} = 100 \times \frac{M_{be}}{M_{mb}}$$

P_{be} is most readily calculated with the following equation:

$$P_{be} = P_b - \frac{P_{ba}}{100} P_s$$

where

P_{be} is effective binder, percentage by mass of mix

Pb is total binder, percentage by mass of mix

P_{ba} is absorbed binder, percentage by mass of aggregate

P_s is total aggregate, percentage by mass of mix

M_{be} is mass of the effective binder, typically in grams

M_{mb} is mass of the total mix, typically in grams

5.9 Dust to binder ratio

The dust to binder ratio ($P_{0.075}/P_{be}$) of a paving mixture, sometimes referred to as the "dust proportion," is the ratio of the percentage of aggregate passing the 0.075-mm (No. 200) sieve (P0.075) to the effective binder (P_{be}). The typical allowable range for this property is 0.6–1.2, with the following exceptions:

a) for 4.75-mm mixes, the allowable range is 0.9–2.0; and

b) for coarse-graded mixes whose gradation plots below the Primary Control Sieve (PCS) on a 0.45 power chart, the allowable range may be increased to 0.8–1.6.

In general, this property addresses the workability of asphalt mixtures. A low $P_{0.075}/P_{be}$ often results in a tender mix, which lacks cohesion and is difficult to compact in the field because it tends to move laterally under the roller. Mixes tend to stiffen as the P0.075 increases, but too much will also result in a tender mix. A mix with a high $P0.075/P_{be}$ will often exhibit a multitude of small stress cracks during the compaction process, called check-cracking. This property is usually calculated for dense-graded mixes only.

5.10 Selecting a design aggregate structure (DAS)

Meeting the minimum VMA requirement at the design binder content is often difficult to achieve. It is not uncommon for a mixture designer to complete a laboratory mixture design only to learn that the VMA does not meet the volumetric criteria specified. Careful analysis of the DAS results provide the designer with valuable information upon which to make design changes or decisions on whether or not to proceed with a full mixture design and performance testing.

The Superpave mix design process discussed in 6.1 originally included a procedure called Selecting a Design Aggregate Structure (DAS) to assess the ability of multiple gradations to meet specified VMA criteria at the design binder content. This process can be used to evaluate different aggregate combinations before or after the initial mixture design is completed. The process can be summarized as an analysis of multiple aggregate combinations at one estimated design binder content during a single laboratory session.

The following step-by-step process is conducted for each trial blend prepared. The basic DAS process is suitable for Marshall and Superpave designed mixtures. Additional steps are required that are specific to gyratory compacted mixtures.

Step 1—Calculate $P_{a\ initial}$

$$P_{a_{initial}} = \frac{G_{mm_{initial}} - G_{mb_{initial}}}{G_{mm_{initial}}}$$

where

$P_{a\ initial}$ is % air voids in the mix at $P_{b\ initial}$

$G_{mb\ initial}$ is bulk specific gravity of the mix at $P_{b\ initial}$

$G_{mm\ initial}$ theoretical maximum specific gravity of the mix at $P_{b\ initial}$

Step 2—Calculate $VMA_{initial}$

$$VMA_{initial} = 100 - \frac{G_{mb_{initial}} \times P_s}{G_{sb}}$$

where

$VMA_{initial}$ is voids in the mineral aggregate at P_b initial

$G_{mb\ initial}$ is bulk specific gravity of the mix at P_b initial

P_s percent is aggregate in the specimen

G_{sb} is bulk specific gravity of the aggregate blend

Step 3—Calculate the $\%G_{mm}$ @ N_{des}– for gyratory compacted mixes only

$$\%G_{mm}@N_{des} = 100 - P_{a_{initial}}$$

where

$\%G_{mm}$ @ N_{des} is percent of theoretical maximum specific gravity at N_{des} using P_b initial

$P_{a\ initial}$ is % air voids in the mix at N_{des} using P_b initial

Step 5—Calculate the Correction Factor for G_{mb} @ N_{ini}–for gyratory compacted mixes only

$$C = \frac{Ht.@N_{des}}{Ht.@N_{ini}}$$

where

C is the correction factor

Ht.@Ndes is a height of the specimen at N_{des}

Ht.@Nini is height of the specimen at N_{ini}

Step 6—Calculate the G_{mb} @ N_{ini}– for gyratory compacted mixes only

$$G_{mb}@N_{ini} = C \times \left(G_{mb}@N_{des}\right)$$

Step 7—Calculate P_b estimated

$$P_{b_{estimated}} = P_{b_{initial}} - \ 0.4 \times \left(4.0 - Va_{initial}\right)$$

where

$P_{b\ estimated}$ is the asphalt content needed to obtain a mixture at the design Pa

$P_{b\ initial}$ is the initial binder content of the trial blend

$P_{a\ initial}$ is percent air voids in the mix at $P_{b\ initial}$

0.4 is a value derived from the slope of the P_a curve which equates to a 0.4% change in binder = 1% change in air voids

4.0 = the design air void level

$P_{b\ estimated}$ is an estimated value of how much binder should have been added to the trial blend to achieve the design air voids in the compacted mixture. The following steps calculate the typica volumetric properties in the mixture assuming that

$P_{b\ estimated}$ had been added to the trial blend.

Step 8—Calculate VMAestimated

$$VMA_{estimated} = VMA_{initial} + \ C \times \left(4.0 - P_{a_{initial}}\right)$$

where

$VMA_{estimated}$ is the estimated VMA had the trial blend used Pb estimated

$VMA_{initial}$ Voids in the mineral aggregate at $P_{b\ initial}$

C (Constant) is equal to 0.1 if Va is less than 4.0% and 0.2 if Va is greater than 4.0%

Pa initial is % air voids in the mix at $P_{b\ initial}$

4.0 is the design air void level

The VMA in a compacted mixture is a result of the aggregate properties, binder properties, temperature and perhaps the most significant, the type and amount of compactive effort exerted on the specimen. Minute changes in binder content in close proximity to the design binder content have little effect on VMA, but are accounted for in the above-referenced C value.

$$VFA_{estimated} = 100 \times \ \frac{\left(VMA_{estimated} - 4.0\right)}{VMA_{estimated}}$$

where

VFAestimated the estimated VFA had the trial blend used Pb estimated

VMAestimated the estimated VMA had the trial blend used

Pb estimated 4.0 is the design air void content

Step 10—Calculate P_{be} estimated

The effective asphalt binder content is calculated using

$$P_{be_{estimated}} = P_{b_{estimated}} - \left(P_s \times G_b\right) \times \frac{G_{se} - G_{sb}}{G_{se} \times G_{sb}}$$

where

Pbe estimated is the estimated effective binder content had the trial blend used Pb estimated

Ps is aggregate content, percent by total mass of the mixture

Gb is specific gravity of the asphalt

Gse is effective specific gravity of the aggregate

Gsb is bulk specific gravity of the aggregate

Pb estimated is the asphalt content needed to obtain a mixture at 4.0% air voids

Step 11—Calculate Dust Proportion

The requirement for the dust proportion is calculated as the percent by mass of the material passing the 0.075-mm sieve (by wet sieve analysis) divided by the effective asphalt binder content (expressed as percent by mass of mix).

$$DP = \frac{P_{0.075}}{P_{be_{estimated}}}$$

where

Pbe estimated is the estimated effective binder content had the trial blend used Pb estimated

P0.075 is aggregate content passing the 0.075-mm sieve

Finally, for Superpave gyratory compacted mixtures:

Step 12—Calculate $\%G_{mm,estimated@Nini}$

$$\%Gmm@Nini_{estimated} = \%Gmm@Nini_{initial} - \left(4.0 - P_{a_{initial}}\right)$$

where

4.0 = the design air void content

6 Asphalt mix design methodologies

6.1 Superpave HMA Mix Design System

The Superpave system as implemented consisted of two interrelated elements: an asphalt binder specification and a mix design system that specifies aggregate criteria and volumetric properties. The Superpave mix design method uses a gyratory compactor to make test specimens and volumetrics to determine the optimum binder content. Moisture resistance, and in some cases, performance testing are included to verify the suitability of the designed mixture. Superpave is a system which rely on the principle that a pavement mixture will not perform successfully on the roadway unless the appropriate binder and aggregate materials are incorporated into the work.

6.1.1 Superpave materials selection and mix design criteria

6.1.1.1 Asphalt binder

The asphalt binder to be used in the mix is chosen based on environmental conditions and traffic loadings. The grade of asphalt binder to be incorporated into the HMA is usually specified during the project design and is not a variable considered during the mix design phase.

6.1.1.2 Mineral aggregate specifications

Superpave mix design system contain detailed aggregate requirements. The detailed mineral aggregates characteristics are described in Annex B. Certain aggregate characteristics were essential to design a well performing HMA. These characteristics were broken into consensus and sources characteristics as outlined below:

a) **The consensus aggregate properties**

These are specified in the Superpave system are the following: coarse aggregate angularity; fine aggregate angularity; flat and elongated particles; and clay content (Sand Equivalent). The criteria for these properties are based on the traffic level and position within the pavement structure. Table 1 gives the requirement for the aggregate consensus properties

Table 1 — Aggregate consensus properties requirement

20-Year Design ESALs[a] (in millions)	Coarse Aggregate Angularity (Percent), minimum[c] AASHTO T 335 (CAA)		Uncompacted Void Content of Fine Aggregate Angularity (Percent), minimum AASHTO T 304 (FAA)		Sand Equivalent (Percent), minimum AASHTO T 176 (SE)	Flat and Elongated[c] (Percent), maximum ASTM D4791 (F&E)
	≤ 100 mm[f]	> 100 mm[f]	≤ 100 mm	> 100 mm[f]		
< 0.3	55/-	-/-	-[d]	-	40	-
0.3 to < 3	75/-	50/-	40[e]	40	40	10
3 to < 10	85/80[b]	60/-	45	40	45	10
10 to < 30	95/90	80/75	45	40	45	10
≥ 30	100/100	100/100	45	45	50	10

NOTES:

a 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} levels.

b 85/80 denotes that 85 percent of the coarse aggregate has one or more fractured faces and 80 percent has two or more fractured faces.

c This criterion does not apply to 4.75-mm nominal maximum aggregate size mixtures.

d For 4.75-mm nominal maximum aggregate size mixture designed for traffic levels below 0.3 million ESALs, the minimum Uncompacted Void Content is 40.

e For 4.75-mm nominal maximum aggregate size mixture designed for traffic levels equal to or above 0.3 million ESALs, the minimum Uncompacted Void Content is 45.

f If less than 25 percent of a construction lift is within 100 mm of the surface, the lift may be considered to be below 100 mm for mix design purposes.

b) Source aggregate properties

These are requirements specific to local areas because of the variety of aggregate types available in different geographic areas, and are not broad, all-inclusive requirements like Superpave consensus properties. Source property tests are typically specified to set limits on aggregate properties mainly toughness, soundness and deleterious materials. Table 2 lists some recommended source property tests and typical requirements.

Table 2 — Recommended superpave source property teste and typical requirements

20-Year Design Equivalent Single Axle Loads (ESALs in millions)	Los Angeles Abrasion (Max. %) AASHTO T 96	Sodium or Magnesium Sulfate Soundness (Max. %) AASHTO T 104	Deleterious materials*	
			Clay Lumps/ Friable Particles AASHTO T 112	Lightweight Particles AASHTO T 113
< 0.3	45	25	<5	<5
0.3 to < 3	40	20	<4	<4
3 to < 10	30	15	<3	<3
10 to < 30	30	15	<2	<2
≥ 30	25	<10	<1	<1
*Specific tests and property requirements to be determined locally				

6.1.1.2.1 Gradation

Superpave graduation requirements are shown in Table 3. Control points have been established to define the type of mix. It is important to note that the recommended gradation control points are limited in number when compared to traditional HMA gradation specifications. The following gradation control points should be regarded as a requirement for mixture design only. The Superpave gradation requirements shown are only useful if the aggregates meet the consensus and source properties and the completed asphalt mixture meets the volumetric and/or performance requirements.

Table 3 — Superpave graduation requirements

NMAS	37.5 mm		25 mm		19 mm		12.5 mm		9.5 mm		4.75 mm	
Sieve, mm	Min. % P	Max. % P	Min. % P	Max. % P	Min. % P	Max. % P	Min. % P	Max. % P	Min. % P	Max. % P	Min. % P	Max. % P
50.0	100	-	-	-	-	-	-	-	-	-	-	-
37.5	90	100	100	-	-	-	-	-	-	-	-	-
25.0	-	90	90	100	100	-	-	-	-	-	-	-
19.0	-	-	-	90	90	100	100	-	-	-	-	-
12.5	-	-	-	-	-	90	90	100	100	-	100	-
9.5	-	-	-	-	-	-	-	90	90	100	95	100
4.75	-	-	-	-	-	-	-	-	-	90	90	100
2.36	15	41	19	45	23	49	28	58	32	67	-	-
1.18	-	-	-	-	-	-	-	-	-	-	30	55
0.600	-	-	-	-	-	-	-	-	-	-	-	-
0.300	-	-	-	-	-	-	-	-	-	-	-	-
0.150	-	-	-	-	-	-	-	-	-	-	-	-
0.075	0.0	6.0	1.0	7.0	2.0	8.0	2.0	10.0	2.0	10.0	6.0	13.0

6.1.1.3 Superpave mixture requirements

6.1.1.3.1 N_{ini}, N_{des}, N_{max}

6.1.1.3.1.1 Table 4 designates specified levels of laboratory compaction (density) at three different levels of gyration based on expected traffic loading and referenced as $N_{initial}$, N_{design} and $N_{maximum}$. These terms, more commonly designated as N_{ini}, N_{des} and N_{max}, refer to the number of gyrations estimated to result in different levels of field densification. N_{des} is the number of gyrations specified to reach the target density of the mix and is based on the estimated field density in the middle of its service life. N_{ini} is a measure of the compactability of the mix and is based on the estimated field density obtained behind the screed before compaction. N_{max} is an estimate of the final density expected in the field at the end of its service life after years of further densification by traffic. It is capped at 98 percent for all traffic levels.

Table 4 — Superpave gyration compaction effort

20-Year Design ESALs* (in millions)	Compaction Parameters			Typical Roadway Applications
	$N_{initial}$	N_{design}	$N_{maximum}$	
< 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 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 highways. Among these are medium to heavily trafficked city streets, many state routes, U.S. highways and some rural interstates.
≥ 30	9	125	205	Applications include the vast majority of the U.S. 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.
*Laboratory compaction parameters should be based on 20-year design ESALs, regardless of the pavement design life of the roadway.				

6.1.1.3.1.2 The number of design gyrations was established by comparing lab-molded densities at different gyration levels to resulting field densities. Once the design gyration levels were established, the initial and maximum gyration levels were established by the following equations:

$$N_{ini} = (N_{des})^{0.45}$$

$$N_{max} = (N_{des})^{1.10}$$

Table 5 lists the specified levels of densificationalong with all of the required Superpave volumetric parameters.

Table 5 — Superpave mixture requirements

20-Year Design ESALs[a] (in millions)	Required Relative Density During Mix Design, Percent of Theoretical Maximum Specify Gravity (% G_{mm})			Voids in the Mineral Aggregate (VMA), Percent Minimum					Voids Filled with Asphalt (VFA)[b] Range, Percent	Dust-to-Binder Ratio (DP) Range[c]
	$N_{initial}$	N_{design}	$N_{maximum}$	Nominal Maximum Aggregate Size (NMAS), mm						
				37.5	25.0	19.0	12.5	9.5		
< 0.3	≤ 91.5	96.0	≤ 98.0	11.0	12.0	13.0	14.0	15.0	70–80 [d,e]	0.6–1.2
0.3 to < 3	≤ 90.5								65–78 [f]	
3 to < 10	≤ 89.0								65–75 [e,f,g]	
10 to < 30										
≥ 30									65–75 [g]	

NOTES:

a 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.

b For 37.5-mm nominal maximum aggregate size mixtures, the specified lower limit of the VFA shall be 64 percent for all design traffic levels.

c For 4.75-mm nominal maximum aggregate size mixtures, the dust-to-binder ratio shall be 1.0 to 2.0, for design traffic levels < 3 million ESALs, and 1.5 to 2.0 for design traffic levels ≥ 3 million ESALs.

d For 4.75-mm nominal maximum aggregate size mixtures, the relative density (as a percent of the theoretical maximum specific gravity) shall be within the range of 94.0 to 96.0 percent.

e For design traffic levels < 0.3 million ESALs and for 25.0-mm nominal maximum size mixtures, the specified lower limit of the VFA range shall be 67 percent, and for 4.75-mm nominal maximum aggregate size mixtures, the specified VFA range shall be 67 to 69 percent.

f For design traffic levels > 0.3 million ESALs and for 4.75-mm nominal maximum aggregate size mixtures, the specified VFA range shall be 66 to 67 percent.

g For design traffic levels ≥ 3 million ESALs and for 9.5-mm nominal maximum aggregate size mixtures, the specified VFA range shall be 73 to 76 percent.

If the aggregate gradation passes beneath the specified PCS Control Point, the dust-to-binder ratio range may be increased from 0.6–1.2 to 0.8–1.6 at the agency's discretion.

Mixtures with VMA exceeding the minimum value by more than 2 percent may be prone to flushing and rutting. Unless satisfactory experience with high VMA mixtures is available, mixtures with VMA greater than 2 percent above the minimum should be avoided.

6.1.2 Test equipment

6.1.2.1 Superpave gyratory compactor

The typical superpave gyration compactor consist of:

a) reaction frame, rotating base and motor;

b) loading system, loading ram and pressure gauge;

c) height measuring and recordation system;

d) mould and base plate; and

e) specimen extruding device.

The main parameters governing the compaction effort are:

a) the vertical pressure, 600 ± 18 kPa;

b) the gyration angle of the mold, 1.16 ± 0.02° (internal);

c) gyration rate, 30.0 ± 0.5 gyrations per minute; and

d) the number of gyrations, variable—based on expected traffic level.

6.1.2.2 Additional test equipment

1) **ovens**—thermostatically controlled, for heating aggregates, asphalt and equipment;

2) **mechanical mixer,** commercial bread dough mixer 10-liter (10-qt.) capacity or larger, equipped with metal mixing bowls and wire whips, or a 5-gallon laboratory bucket mixer;

3) flat-bottom metal pans for heating aggregates and conditioning mixtures;

4) **round metal pans**—approximately 10-liter capacity for mixing asphalt and aggregate;

5) **scoops for batching aggregates**;

6) **containers,** gill-type tins, beakers or pouring pots, for heating asphalt;

7) **thermometers or other thermometric devices,** armored, glass or dial-type with metal stem, 10°C to 235°C, for determining temperature of aggregates, asphalt and asphalt mixtures;

8) **balances,**10-kg capacity, sensitive to 1 g, for weighing aggregates and asphalt; 10-kg capacity, sensitive to 0.1 g, for weighing compacted specimens;

9) **large mixing spoon or small trowel**;

10) large spatula;

11) welders gloves (or similar) for handling hot equipment; 12. paint, markers or crayons, for identifying test specimens;

12) paper disks, 150 mm, for compaction;

13) fans, for cooling compacted specimens; and

14) computer/printer for data collection and recording.

6.1.3 Specimen preparation and compaction

6.1.3.1 Preparation of aggregate

The following samples must be prepared to conduct a Superpave mix design:

a) a minimum of 8 specimens to be compacted to $N_{des;}$

b) a minimum of 2 specimens for maximum theoretical specific gravity (G_{mm}) testing;

c) a minimum of 6 specimens with a height of 95 mm. These samples require approximately 3,700 grams of aggregate for moisture sensitivity testing.

d) a minimum of 2 specimens for N_{max} verification; and

e) any other samples required for specified performance testing.

Three aggregate sample sizes are used in conducting a Superpave mix design, depending on their final use. For compacted G_{mb} samples, the specimen size is 150 mm (diameter) by 115 mm (height) and requires two samples of approximately 4,700 grams of aggregate to be prepared for each trial asphalt content tested plus 2 additional specimens for N_{max} verification testing.

A minimum of two samples need to be prepared for determination of maximum theoretical specific gravity (G_{mm}). These samples remain uncompacted, and their size varies by the nominal maximum aggregate size (NMAS) of the mixture and ranges from 1,000 to 2,500 grams. These samples will be mixed at an asphalt binder content at or above the final estimated design binder content. The G_{mm} values for the remaining trial binder contents may be back-calculated according to the equations shown in clause 5.

6.1.3.2 Preparation of binder

Samples of asphalt binder to be used in the mix design shall be representative of the materials to be used in the project and should be of a size that is manageable in the laboratory. Care should be taken not to overheat the binder nor maintain elevated temperatures for an extended period of time. The mixture design will require the molding of mixture specimens at a minimum of four different binder contents. It is recommended that specimens be molded at the anticipated design binder content and at –0.5, +0.5 and +1.0 percent binder bracketing the anticipated design binder content.

6.1.3.3 Laboratory mixing and compaction temperatures

The mixing and compaction temperatures shall be determined according to Annex A. Preheat the aggregate samples in an oven set approximately 15°C higher than the mixing temperature. Two to four hours are required for the aggregate to reach the mixing temperature. Preheat the asphalt binder to the predetermined mixing temperature.

The time required for this step varies depending on the container size and method of heating. Care should be taken to adjust the preheating time in a manner that does not expose the binder to elevated temperatures for an extended period of time.

6.1.3.4 Preparation of mixtures

a) Place the hot mixing bowl on a balance and zero the balance.

b) Charge the mixing bowl with the heated aggregates and mix thoroughly.

c) Form a crater in the blended aggregate and weigh the required asphalt in the mixture to achieve the desired batch weight.

d) Remove the mixing bowl from the scale and mix the asphalt and aggregate using a mechanical mixer.

e) Mix the specimen until the aggregate is thoroughly coated (30–90 seconds).

f) Place the mix in a flat shallow pan at an even thickness ranging between 25 and 50 mm.

g) Place the mix and pan in the conditioning oven at a temperature equal to mixture's specified compaction temperature ± 3°C.

h) Condition the prepared mixture in accordance with (Annex A)

i) Repeat this procedure until the desired number of specimens is produced.

j) Proper timing of the gyratory compaction steps can be achieved by spacing approximately 20 minutes between mixing each specimen.

k) At the end of the short-term aging period, proceed to ASTM D2041 if the mixture is to be used to determine maximum theoretical specific gravity. Otherwise, proceed with compaction.

6.1.3.5 Compaction of volumetric specimens

6.1.3.5.1 Molding procedure for specimens at a specified number of gyrations

6.1.3.5.1.1 Prepare at least two replicate specimens for each trial binder content at the N_{des} gyration level. G_{mb} specimen heights after compaction should be 115 ± 5 mm. Generally, placing approximately 4,700 grams of mixture into the mold for blends with aggregate specific gravities between 2.55 and 2.70 will result in a compacted specimen meeting height requirements.

6.1.3.5.1.2 Prepare the SGC while the mixture specimens are conditioning in the oven. Turn on the main power to the SGC for the manufacturer's recommended warm-up period. Further preparations include verifying the compaction pressure of 600 kPa, verifying the gyration internal angle of 1.16°, and verifying the gyration speed of 30 revolutions per minute. Set the SGC to the proper number of gyrations according to Table 4. Lubricate any bearing surfaces according to manufacturer instructions.

6.1.3.5.1.3 Approximately 45 to 60 minutes before compaction of the first specimen, place the molds and base/top plates in an oven set at the compaction temperature. Remove the mold and base plate from the oven, place the base plate inside the mold and place a paper disk (to prevent the mix from sticking to the base plate) on top of the base plate.

6.1.3.5.1.4 Place the conditioned mixture at the compaction temperature in the mold in one lift using a material handling chute, taking care to avoid segregation of the mix. Level the mix, place another paper disk on top of the mix and place the top plate (if required) in the mold. Load the charged mold into the SGC, centering it under the loading ram.

6.1.3.5.1.5 Depending on the SGC make and model, the following steps will either be performed automatically after pressing a "start" button or be initiated by the user:

a) Lower the loading ram until it reaches a pressure of 600 kPa;

b) Apply the 1.16° angle; and

c) Proceed with gyrations to the preset number.

During compaction, the ram loading system will maintain a constant pressure of 600 kPa. The specimen height will be continually monitored during compaction, and a height measurement is recorded after each gyration.

The compactor will cease compacting after reaching the user-specified number of gyrations, the angle of gyration will be released and the loading ram will be raised. Remove the mold containing the compacted

specimen from the compactor and slowly extrude the specimen from the mold. Most mixes can be removed immediately, but some mixes may require a 5- to 10-minute cooling period before they can be removed without distortion.

6.1.3.5.1.6 Remove the paper disks from the top and bottom of the specimen and then allow the specimen to cool on a flat surface, undisturbed. Place the mold and base/top plates back in the oven to reach compaction temperature for the next specimen. Additional molds will avoid the delay caused by this step.

6.1.3.5.1.7 Repeat the compaction procedure for each specimen. Identify each specimen with a suitable marker.

6.1.3.5.2 Molding procedure for specimens at a specified height

6.1.3.5.2.1 The actual molding procedure is the same, except that the SGC is set to "height" mode instead of "Number of gyrations" mode. This will cause the molding process to stop after reaching the specified height, rather than stopping after a specified number of gyrations.

6.1.3.5.2.2 Some procedures, call for specimens to be molded to a specific height with a specific percentage of air voids. To accomplish this, the designer must determine the appropriate mass of asphalt mixture to place in the mold that will result in the proper air void content at the proper height.

6.1.3.5.2.3 There are multiple ways to make this determination. One method requires the designer to mix and mold two extra specimens. A slightly higher-than-the-finalanticipated mass is placed in one mold and a slightly lower-than-the-final anticipated mass is placed in the other. After each specimen is molded to the specified height, the first specimen will have slightly lowerthan- target air voids and the second specimen will have slightly higher-than-target air voids. The designer can then interpolate the masses to achieve the proper air voids.

6.1.4 Superpave data analysis

6.1.4.1 Determine $\%G_{mm}$ at N_{ini} for each binder percentage

An additional step unique to Superpave is the calculation of $\%G_{mm}$ at N_{ini}, N_{des} and N_{max}. Using the data provided by the SGC, an estimated mixture bulk specific gravity, $G_{mb,estimated}$, can be calculated based on the diameter of the mold and the height of the specimen. This $G_{mb,estimated}$ value can be calculated for every gyration. The final product of the compaction process is an actual specimen with known height at N_{des}.

Step 1—Obtain the height of the Specimen at N_{des} (h_{Ndes}).

Step 2—Obtain the height of the Specimen at N_{ini} (h_{Nini}).

Estimating the $G_{mb,estimated}$ at N_{ini} using the smooth mold dimensions would not account for surface imperfections in the sample. These surface imperfections are accounted for when the sample compacted to N_{des} is actually weighed in the lab. By using the following correction factor, the estimated G_{mb} at N_{ini} can be more accurately calculated.

Step 3—Calculate the correction factor C, as follows:

$$C = h_{Ndes} / h_{Nini}$$

Step 4—Calculate $G_{mb,\ estimated}$ @ N_{ini}

$$G_{mb,\ est} @\ N_{ini} = C \times G_{mb} @\ N_{des}$$

Step 5—Determine the average G_{mb} @ N_{ini} value for each trial binder content.

Step 6—Calculate %G_{mm} values as follows:

$$\%G_{mm} = \frac{G_{mb} @ N_{ini}}{G_{mm}\ (at\ the\ trial\ P_b)} \times 100$$

or

$$\%G_{mm} = \frac{G_{mb} @ N_{des}}{G_{mm}\ (at\ the\ trial\ P_b)} \times 100$$

or

$$\%G_{mm} = \frac{G_{mb} @ N_{max}}{G_{mm}\ (at\ the\ trial\ P_b)} \times 100$$

6.1.4.2 Superpave volumetric analysis

The remaining volumetric properties for P_a, VMA, VFA, P_{be} and DP are calculated as shown in Clause 5. The results are plotted to provide curves of asphalt content versus $\%G_{mm}$ @ N_{ini}, % G_{mm} @ N_{des}, air voids (P_a), voids in the mineral aggregate (VMA), voids filled with asphalt (VFA) and dust proportion (DP).

6.1.5 Design asphalt binder content

After the data have been plotted, the designer will pick an asphalt binder content that will provide 4.0 percent air voids at N_{des}, which is equal to 96.0 percent $G_{mm@Ndes}$. The designer must then verify the mixture conforms to the specifications by determining the values for percent $G_{mm@Nini}$, voids in the mineral aggregate (VMA), voids filled with asphalt (VFA) and dust proportion (DP) from the other curves. The recommended Superpave volumetric mixture design requirements are shown in Table 5

6.1.6 Nmax determination

The N_{max} parameter must be determined and verified that the $G_{mm@Nmax}$ does not exceed 98 percent. Early versions of the Superpave mix design system required designers to compact all of the mixture specimens discussed above to N_{max}.The values for N_{ini} and N_{des} are then both back-calculated as was discussed for N_{ini}.

The average of these two sets of data will be used to directly calculate the $\%G_{mm@Nmax}$. Verify that the result is less than 98 percent of the G_{mm} at the design binder content.

6.1.7 Moisture sensitivity testing

The next step in the mixture design process is to conduct moisture sensitivity testing. The mixture must meet all of the aggregate and volumetric criteria prior to proceeding. All samples prepared for performance testing will be mixed at the design binder content determined. The Superpave system requires that the designed mixture meet the specified moisture sensitivity requirements.

6.1.8 Performance testing

Although not a formal requirement of the Superpave design system, it is recommended that strong consideration be given to conducting some type of additional mixture performance testing, especially on critical, high volume projects.

6.2 Marshall Method of Mix Design

6.2.1 General

The Marshall method of mix design is for dense graded HMA mixes. For a single selected aggregate gradation, five different asphalt contents are tested for various volumetric and strength criteria to select the optimum binder content. The test results should always be reported as the average for three compacted, “identical” specimens. The selection of the optimum binder content requires engineering judgment, depending on traffic, climate and experience with the local materials used. In most cases, the optimum binder content should be selected for which the compacted specimens have 4 percent air voids.

The original Marshall method is applicable only to hot mix asphalt paving mixtures containing aggregates with maximum sizes of 25 mm or less. A modified Marshall method has been developed for aggregates with maximum sizes up to 38 mm.

6.2.3 Outline of method

The procedure for the Marshall method starts with the preparation of test specimens. Steps preliminary to specimen preparation are:

a) all materials proposed for use shall meet the physical requirements of the project specifications;

b) aggregate blend combinations shall meet the graduation requirements of the project specifications; and

c) for the purpose of performing density and voids analyses, the bulk specific gravity of all aggregates used in the blend and the specific gravity of the asphalt cement are determined.

These requirements are matters of routine testing, specifications and laboratory technique that must be considered for any mix design method.

The Marshall method uses standard test specimens of 63.5-mm height by a 101.6-mm diameter. These are prepared using a specified procedure for heating, mixing, and compacting the asphalt-aggregate mixture. The two principle features of the Marshall method of mix design are

a) a density-voids analysis and

b) a stability-flow test of the compacted test specimens.

6.2.4 Preparation of test specimens

6.2.4.1 General

6.2.4.1.1 In determining the design asphalt content for a particular blend or gradation of aggregates by the Marshall method, a series of test specimens is prepared for a range of different asphalt contents so that the test data curves show well-defined relationships. Tests should be planned on the basis of 0.5 percent increments of asphalt content, with at least two asphalt contents above the expected design value and at least two below this value.

6.2.4.1.2 The expected design asphalt content can be based on any or all of these sources: experience, computational formula, or performing the centrifuge kerosene equivalency and oil soak tests in the Hveem procedure (see 6.3). The expected design asphalt content, in percent by total weight of mix, could then be estimated to be approximately equivalent to the percentage of aggregate in the final gradation passing the 75-µm (No. 200) sieve.

One example of a computational formula is this equation:

$P = 0.035a + 0.045b + Kc + F$

where

P is approximate asphalt content of mix, percent by weight of mix

a is percent of mineral aggregate retained on 2.36-mm (No. 8) sieve

b is percent of mineral aggregate passing the 2.36-mm (No. 8) sieve and retained on the 75-µm (No. 200) sieve

c is percent of mineral aggregate passing 75-µm (No. 200) sieve

K 0.15 for 11–15 percent passing 75-µm (No. 200) sieve, 0.18 for 6–10 percent passing 75-µm (No. 200) sieve, 0.20 for 5 percent or less passing 75-µm (No. 200) sieve

F is 0 – 2.0 percent. Based on the absorption of light or heavy aggregate, in the absence of other data, a value of 0.7 is suggested.

6.2.4.1.3 To provide adequate data, at least three test specimens are prepared for each asphalt content selected. Therefore, a Marshall mix design using five different asphalt contents will normally require at least 15 test specimens. Each test specimen will usually require approximately 1.2 kg of aggregate. Assuming some minor waste, the minimum aggregate requirements for one series of test specimens of a given blend and gradation will be approximately 23 kg. About 4 liters of asphalt cement will be adequate.

6.2.4.2 Equipment

The equipment (calibrated as needed) required for the preparation of test specimens is:

1) flat-bottom metal pans for heating aggregates;

2) round metal pans or a mixing bowl, approximately 4-liter capacity, for mixing asphalt and aggregate; oven and hot plate, preferably thermostatically controlled, for heating aggregates, asphalt and equipment;

3) scoop for batching aggregates;

4) **containers:** gill-type tins, beakers, pouring pots and sauce pans for heating asphalt;

5) thermometers or other thermometric devices: armored, glass or dial-type with metal stem, 10°C to 235°C, for determining temperature of aggregates, asphalt and asphalt mixtures;

6) balances: 5-kg capacity, sensitive to 1 g, for weighing aggregates and asphalt, and 2-kg capacity, sensitive to 0.1 g, for weighing compacted specimens;

7) large mixing spoon or small trowel;

8) large spatula;

9) **mechanical mixer** (optional): commercial bread dough mixer 4-liter capacity or larger, equipped with two metal mixing bowls and two wire stirrers, or an equivalent type mixer.

10) compaction pedestal (shown in Figure 2), consists of a 200 × 200 × 460 mm wooden post capped with a 305 × 305 × 25 mm (12 × 12 × 1 in.) steel plate. The wooden post should be oak, pine or other wood having a dry weight of 670 to 770 kg/m3. The wooden post should be secured by four angle brackets to a solid concrete slab. The steel cap should be firmly fastened to the post. The pedestal should be installed so that the post is plumb, the cap level, and the entire assembly free from movement during compaction. Compaction hammers can be either manually or mechanically operated, as shown in Figure 2. Mechanically operated hammers drop the hammer at a rate of 64 ± 4 blows per minute. Mechanical hammers can also have single or multiple hammer and mold sets for compacting single or multiple specimens at a time. Some mechanically operated hammers are designed with a rotating base mechanism which rotates at 18 to 30 revolutions per minute;

Figure 2 — Manual and Mechanical Hammer Configurations

11) compaction mold, consisting of a base plate, forming mold and collar extension. The forming mold has an inside diameter of 101.6 mm and a height of approximately 75 mm; the base plate and collar extension are designed to be interchangeable with either end of the forming mold. with a 114.3-mm height

12) compaction hammer, consisting of a flat, circular tamping face, 98.4 mm in diameter and equipped with a 4.5-kg weight constructed to obtain a specified 457- mm height of drop, should conform to requirements of ASTM D6926;

13) mold holder, consisting of spring-tension device designed to hold compaction mold centered in place on the compaction pedestal, should conform to requirements of ASTM D6926;

14) paper disks, 100 mm;

15) steel specimen extractor, in the form of a jack and a disk with a diameter not less than 100 mm and 13 mm thick for extruding compacted specimen form mold.

16) The steel specimen extractor for the 6-inch mold is 151.5 to 152.5 mm in diameter and 13 mm thick;

17) welders' gloves, for handling hot equipment. Rubber gloves for removing specimens from water bath; and

18) marking crayons, for identifying test specimens.

6.2.4.3 Preparation of test specimen

These steps are recommended steps for preparing Marshall test specimens

a) **Number of specimens**—prepare at least three specimens for each combination of aggregates and binder content.

b) **Preparation of aggregates**—dry aggregates to constant weight at 105°C to 110°C and separate the aggregates by dry sieving into the desired size fractions. These size fractions are recommended:

   1) 25.0 to 19.0 mm

   2) 19.0 to 9.5 mm

   3) 9.5 to 4.75 mm

   4) 4.75 to 2.36 mm

   5) Passing 2.36 mm

c) **Determination of mixing and compaction temperature**—the temperature to which the asphalt must be heated to produce viscosities of 170 ± 20 centistokes kinematic and 280 ± 30 centistokes kinematic shall be established as the mixing temperature and compaction temperatures, respectively. These temperatures can be estimated from a plot of the viscosity (log-log centistokes scale) versus temperature (log degrees Rankine scale, °R = °F + 459.7) relationship for the asphalt cement to be used.

   For mixing and compaction temperatures for modified binders, refer to Annex A.

d) **Preparation of mold and hammer**—thoroughly clean the specimen mold assembly and the face of the compaction hammer and heat them in a water bath or on the hot plate to a temperature between 95 and 50°C.

e) **Preparation of mixtures**—weigh into separate pans for each test specimen the amount of each aggregate size fraction required to produce the required gradation and a batch that will result in a compacted specimen 63.5 ± 1.27 mm in height. This will normally be about 1.2 kg.

   It is generally desirable to prepare a trial specimen prior to preparing the aggregate batches. If the trial specimen height falls outside the height limits, the amount of aggregate used for the specimen should be adjusted using:

   for International System of Units (SI),

$$\text{Adjusted mass of aggregate} = \frac{63.5 \times (\text{mass of aggregate used})}{\text{Specimen height (mm) obtained}}$$

   for U.S. Customary Units,

$$\text{Adjusted mass of aggregate} = \frac{2.5 \times (\text{mass of aggregate used})}{\text{Specimen height (in.) obtained}}$$

   Place the pans in the oven or on the hot plate and heat to a temperature not exceeding 28°C above the mixing temperature specified in (c). (If a hot plate is used, provision should be made for dead space, baffle plate or a sand bath beneath the pans and the hot plate to prevent local overheating.)

Charge the mixing bowl with the heated aggregates and dry mix thoroughly. Form a crater in the dry blended aggregate and weigh the required amount of asphalt cement into the mixture in accordance with the calculated batch weights. At this point the temperature of the aggregate and the asphalt must be within the limits of the mixing temperatures established in paragraph (c). Asphalt cement should not be held at mixing temperatures for more than one hour before using. Mix the aggregate and asphalt cement, preferably with a mechanical mixer or by hand with a trowel, as quickly and thoroughly as possible to yield a mixture having a uniform distribution of asphalt.

The standards requirements recommend that Marshall mixes be conditioned according to AASHTO R30.

f) **Packing the mold**—place a filter or non-absorbent paper disk cut to size in the bottom of the mold. Place the entire batch in the mold with collar, and then spade the mixture vigorously with a heated spatula or trowel 15 times around the perimeter and 10 times over the interior. Smooth the surface to a slightly rounded shape. The temperature of the mixture immediately prior to compaction shall be within the limits of the compaction temperature established in paragraph (c); otherwise, it shall be discarded. In no case shall the mixture be reheated.

g) **Compaction of specimens**—place a paper disk on top of the mix and place the mold assembly on the compaction pedestal in the mold holder. As specified according to the design traffic category (see Table 7), apply 35, 50 or 75 blows with the compaction hammer using a free fall of 457 mm. Hold the axis of the compaction hammer as nearly perpendicular to the base of the mold assembly as possible during compaction. Remove the base plate and collar, and reverse and reassemble the mold. Apply the same number of compaction blows to the face of the reversed specimen. After compaction, remove the base plate and the paper disks and allow the specimen to cool at room temperature until no deformation will result when removing it from the mold. When more rapid cooling is desired, electric fans may be used, but not water unless the specimen is in a plastic bag. Remove the specimen from the mold by means of an extrusion jack or other compression device, and then place on a smooth, level surface until ready for testing. Normally, specimens are allowed to cool overnight.

6.2.5 Conducting Marshall Test

6.2.5.1 General

In the Marshall method, each compacted test specimen is subjected to these tests and analysis in the order listed:

a) specimen height determination;

b) bulk specific gravity determination;

c) density and voids analysis; and

d) stability and flow test.

6.2.5.2 Bulk specific gravity determination

The bulk specific gravity test may be performed as soon as the freshly compacted specimens have cooled to room temperature. This test is performed according to ASTM D1188 when using Paraffin-Coated Specimens and ASTM D2726, when using Saturated Surface-Dry Specimens.

6.2.5.3 Equipment for stability and flow tests

The equipment required for the testing of the 101.6-mm diameter by 63.5-mm height specimens is as follows:

• **The Marshall testing machine** is a compression testing device conforming to ASTM D6927. It is designed to apply loads to test specimens through cylindrical segment testing heads [inside radius of curvature of 51 mm] at a constant loading rate of 51 mm) per minute. Two perpendicular guide posts are included to allow the two segments to maintain horizontal positioning and free vertical movement during the test. It is equipped with

a calibrated proving ring for determining the applied testing load, a Marshall stability testing head for use in testing the specimen, and a Marshall flow meter (or automatic recording device) for determining the amount of deformation at the maximum load in the test. A universal testing machine equipped with suitable load and deformation indicating devices may be used instead of the Marshall testing frame.

• **The water bath** must be at least 150 mm deep and thermostatically controlled to 60°C ± 1°C. The tank should have a perforated false bottom or be equipped with a shelf for suspending specimens at least 50 mm above the bottom of the bath.

6.2.5.4 Stability and flow test procedures

After the bulk specific gravity of the test specimens has been determined, the following stability and flow tests are performed:

(a) determination of specimen height;

(b) immerse specimen in water bath at 60°C ± 1°C for 30 to 40 minutes before testing, or in an oven at the same temperature for 120 to 130 minutes; and

(c) use an automatic recording device, or use a proving ring and flow meter. Place the flow meter over the marked guide rod and “zero” the flow meter while holding it firmly against the upper segment of the testing head while the load is being applied.

NOTE The same assembly of the testing head and flow meter must be used in testing all specimens. Specimens should be 101.6 ± 0.25 mm. Otherwise, an initial and final reading of the flow meter is required for the determination of the flow value.

(d) Thoroughly clean the inside surfaces of the testing heads. Temperature of heads shall be maintained between 21.1 and 37.8°C using a water bath, when required. Lubricate guide rods with a thin film of oil so that the upper test head will slide freely without binding. If a proving ring is used to measure applied load, check to see that the dial indicator is firmly fixed and “zeroed” for the “no-load” position.

(e) With the testing apparatus ready, remove the test specimen from water bath and carefully dry surface with a towel. Place specimen in lower testing head and center; then fit upper testing head into position and center complete assembly in loading device. Place flow meter over marked guide rod as noted in (c) above.

(f) Apply testing load to specimen at a constant rate of deformation, 51 mm per minute, until failure occurs. The point of failure is defined when the maximum load reading is obtained. The total force in Newtons (N) required to produce failure of the specimen shall be recorded as its Marshall stability value.

(g) While the stability test is in progress (if not using an automatic recording device), hold the flow meter firmly in position over the guide rod and remove immediately when the load begins to decrease, take reading and record.

This reading is the flow value for the specimen, expressed in units of 0.25 mm. For example, if the specimen deformed 3.8 mm, the flow value is 15.

(h) The entire procedure for both the stability and flow measurements, starting with the removal of the specimen from the water bath, shall be completed within a period of 30 seconds.

(i) The Marshall stability is corrected for specimens with a height different than 63.5 mm (see Table 7).

Table 7 — Stability Correlation Ratios

Volume of Specimen, cm^3	Approximate Thickness of Specimen mm _____	in. _____	Correlation Ratio
200 to 213	25.4	1	5.56
214 to 225	27.0	1 1/16	5.00
226 to 237	28.6	1 1/8	4.55
238 to 250	30.2	1 3/16	4.17
251 to 264	31.8	1 1/4	3.85
265 to 276	33.3	1 5/16	3.57
277 to 289	34.9	1 3/8	3.33
290 to 301	36.5	1 7/16	3.03
302 to 316	38.1	1 1/2	2.78
317 to 328	39.7	1 9/16	2.50
329 to 340	41.3	1 5/8	2.27
341 to 353	42.9	1 11/16	2.08
354 to 367	44.4	1 3/4	1.92
368 to 379	46.0	1 13/16	1.79
380 to 392	47.6	1 7/8	1.67
393 to 405	49.2	1 15/16	1.56
406 to 420	50.8	2	1.47
421 to 431	52.4	2 1/16	1.39
432 to 443	54.0	2 1/8	1.32
444 to 456	55.6	2 3/16	1.25
457 to 470	57.2	2 1/4	1.19
471 to 482	58.7	2 5/16	1.14
483 to 495	60.3	2 3/8	1.09
496 to 508	61.9	2 7/8	1.04
509 to 522	63.5	2 1/2	1.00
523 to 535	65.1	2 9/16	0.96
536 to 546	66.7	2 5/8	0.93
547 to 559	68.3	2 11/16	0.89
560 to 573	69.8	2 3/4	0.86
574 to 585	71.4	2 13/16	0.83
586 to 598	73.0	2 7/8	0.81
599 to 610	74.6	2 15/16	0.78
611 to 625	76.2	3	0.76

NOTES:
1. The measured stability of a specimen multiplied by the ratio for the thickness of the specimen equals the corrected stability for a 63.5-mm (2.5-in.) specimen.
2. Volume-thickness relationship is based on a specimen diameter of 101.6 mm (4 in.).

6.2.5.5 Density and voids analysis

After the completion of the stability and flow test, a density and voids analysis is made for each series of test specimens. The calculations for the voids analysis are fully described in Clause 5.

(a) Average the bulk specific gravity values for all test specimens of a given asphalt content; values obviously in error shall not be included in the average. The average value of bulk specific gravity for each binder content shall be used in further computations of voids data.

(b) Determine the average unit weight for each asphalt content by multiplying the average bulk specific gravity value by the density of water (1,000 kg/m3).

(c) Determine the theoretical maximum specific gravity (G_{mm} by ASTM D2041) for at least two asphalt contents, preferably on mixes at or near the design asphalt content. An average value for the effective specific gravity of the total aggregate is then calculated from these values. This value may then be used for calculation of the maximum specific gravity of mixtures with different asphalt contents.

(d) Using the effective (G_{se}) and bulk specific gravity (G_{sb}) of the total aggregate, the average bulk specific gravities of the compacted mix (G_{mb}), the specific gravity of the asphalt (G_b), and the maximum specific gravity of the mix (G_{mm}) determined above in (c), calculate the percent absorbed asphalt (P_{ba}) by weight of dry aggregate, percent air voids (Pa), percent voids filthe mineral aggregate (VMA). These values and calculations are more fully described in clause 5.

6.2.6 Interpretation of test data

6.2.6.1 Preparation of test data

Prepare the stability and flow values and void data.

a) Measured stability values for specimens that depart from the standard 63.5 mm thickness shall be converted to an equivalent 63.5 mm value by means of a conversion factor. Applicable correlation ratios to convert the measured stability values are set forth in Table 7 Note that the conversion may be made on the basis of either measured thickness or measured volume.

b) Average the flow values and the final converted stability values for all specimens of given asphalt content. Values that are obviously in error shall not be included in the average.

c) Prepare separate graphical plots for the following values and connect plotted points with a smooth curve that obtains the “best fit” for all values, as illustrated in Figure 3.

    1) percent air voids (Pa) versus asphalt content;

    2) percent voids in mineral aggregate (VMA) versus asphalt content;

    3) percent voids filled with asphalt (VFA) versus asphalt content;

    4) unit weight of total mix versus asphalt content;

    5) stability versus asphalt content; and

    6) flow versus asphalt content.

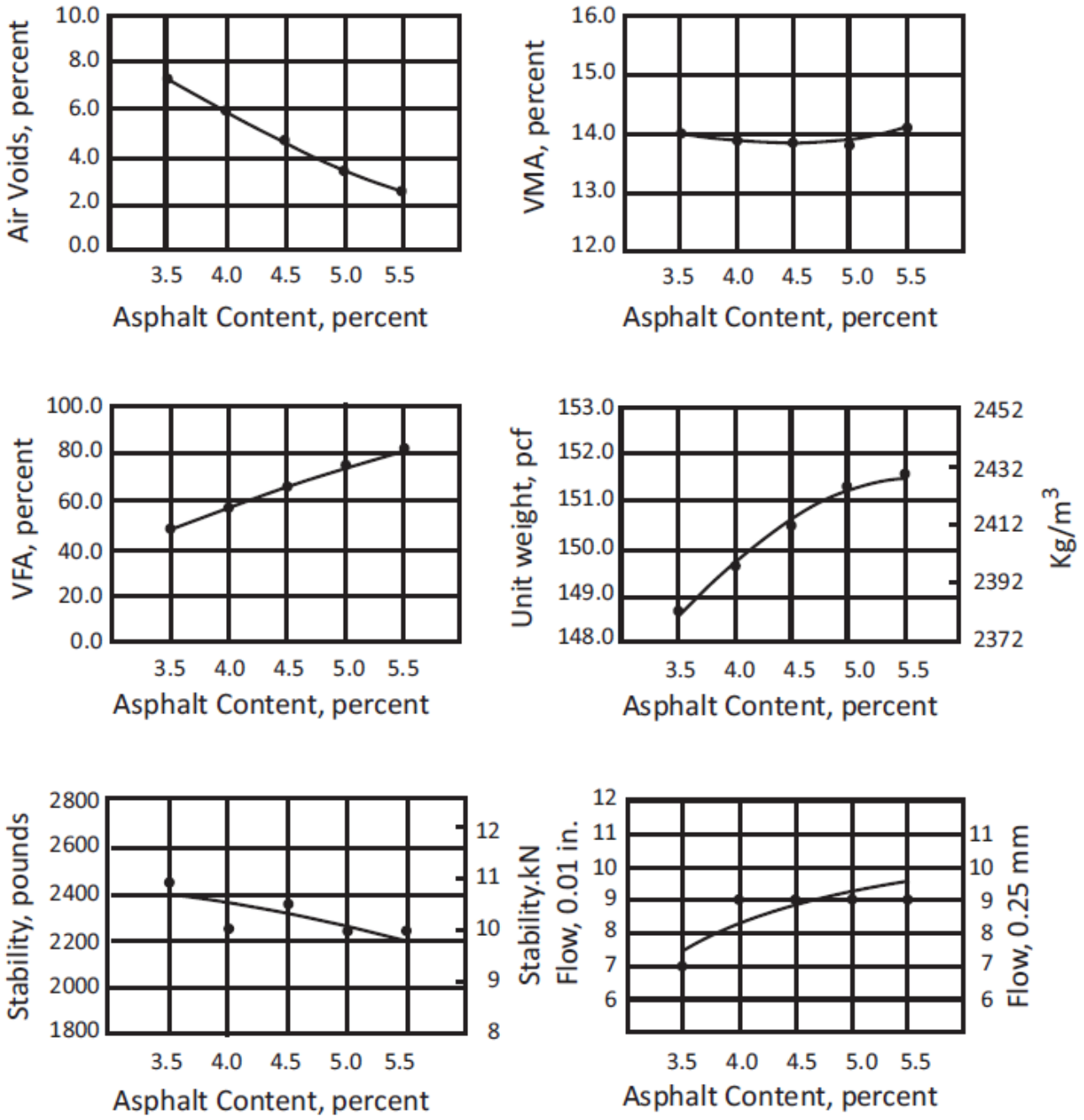

Figure 3 — Test property curves for hot mix design data by the Marshall method

d) Determine the optimum binder content and properties of the mix by using these graphs, as further discussed in 7.2.5.4.

6.2.6.2 Criteria for a satisfactory paving mix

Deciding whether the asphalt paving mix will be satisfactory at the selected design asphalt content is guided by applying certain limiting criteria to the mixture test data. The Marshall method mix design criteria in Table 7 are recommended by the Asphalt Institute.

6.2.6.3 Determination of preliminary design asphalt content

The optimum binder content of the asphalt paving mix is selected by considering all of the data discussed previously. In general, it is recommended to choose the asphalt content at the median of the percent air voids limits, which is 4 percent. If all of the calculated and measured mix properties at this asphalt content meet the mix design criteria in Table 7, then this is the optimum binder content for the mix design.

However, if not all of the design criteria are met, then some adjustment or compromise is necessary or the mix may need to be redesigned (see Footnote 7 of Table 7). A number of considerations are discussed in 6.2.5.4 that should be weighed even if all the design criteria are met.

6.2.6.4 Selection of design binder content

The final selected mix design should be one whose aggregate structure and binder content, compacted to the design number of blows, results in 4 percent air voids and satisfactorily meets all of the other established criteria in Table 7. Deviations from the recommended design criteria should be clearly specified in the project documents and must be appropriate for the intended use of the asphalt mixture. The mixture should contain as much asphalt binder as possible to maximize durability, while also maintaining the stability required to support the intended loads for the life of the pavement.

If the recommended design criteria are modified, experienced designers are encouraged to carefully evaluate project variables such as traffic type and loading, location, materials and climatic region. Two common methods of modifying the mixture design parameters are changing the target air void content and/or changing the laboratory compaction level.

EXAMPLE The FAA P-401 specifies a 3.5 percent design air void content to maximize durability, and it also specifies a compaction level of 75 blows for heavy aircraft along with stringent volumetric parameters to ensure the necessary stability.

A usage with much lighter loads, such as a bike path, may specify 3 percent air voids and reduce the compaction to only 35 blows for increased durability. For a discussion on how to evaluate such parameters as VMA, compaction level, air voids and VFA, reference is made to clause 5.

Table 7 — Marshall mix design criteria

Marshall Method Criteria[1]	Light Traffic[3] Surface & Base		Medium Traffic[3] Surface & Base		Heavy Traffic[3] Surface & Base	
	Min	Max	Min	Max	Min	Max
Compaction, number of blows each end of specimen	35		50		75	
Stability[2], N (lb.)	3336 (750)	–	5338 (1200)	–	8006 (1800)	–
Flow[2,4,5], 0.25 mm (0.01 in.)	8	18	8	16	8	14
Percent Air Voids[7]	3	5	3	5	3	5
Percent Voids in Mineral Aggregate (VMA)[6]	See Table 7.3					
Percent Voids Filled With Asphalt (VFA)	70	80	65	78	65	75

NOTES:

1. All criteria, not just stability value alone, must be considered in designing an asphalt paving mix.
2. Hot mix asphalt bases that do not meet these criteria when tested at 60°C (140°F) are satisfactory if they meet the criteria when tested at 38°C (100°F) and are placed 100 mm (4 inches) or more below the surface. This recommendation applies only to regions having a range of climatic conditions similar to those prevailing throughout most of the United States. A different lower test temperature may be considered in regions having more extreme climatic conditions.
3. Traffic classifications
   Light Traffic conditions resulting in a 20-year Design ESAL < 10^4
   Medium Traffic conditions resulting in a 20-year Design ESAL between 10^4 and 10^6
   Heavy Traffic conditions resulting in a 20-year Design ESAL > 10^6
4. The flow value refers to the point where the load begins to decrease. When an automatic recording device is used, the flow should be corrected as shown in section 7.3.3.3.
5. The flow criteria were established for neat asphalts. The flow criteria are often exceeded when polymer-modified or rubber-modified binders are used. Therefore, the upper limit of the flow criteria should be waived when polymer-modified or rubber-modified binders are used.
6. Percent voids in the mineral aggregate are to be calculated on the basis of the ASTM bulk specific gravity for the aggregate, as discussed in chapter 5.
7. Percent air voids should be targeted at 4 percent. This may be slightly adjusted if needed to meet the other Marshall criteria.

6.2.7 Modified Marshall method for large aggregate

A modified Marshall method for 152.4 mm diameter specimens has been standardized by ASTM D5581 for mixes composed of aggregates with maximum size up to 38 mm. The procedure is basically the same as the original Marshall mix design method except for these differences that are due to the larger specimen size:

a) The hammer weighs 10.2 kg and has a 149.4-mm flat tamping face. Only a mechanically operated device is used for the same 457-mm drop height.

b) The specimen has a 152.4 mm diameter by 95.2 mm height.

c) The batch weights are typically 4,050 g.

d) The equipment for compacting and testing (molds and breaking heads) are proportionately larger to accommodate the larger specimens.

e) The mix is placed in the mold in two approximately equal increments, with spading performed after each increment.

f) The number of blows needed for the larger specimen is 1.5 times (75 or 112 blows) of that required for the smaller specimen (50 or 75 blows) to obtain equivalent compaction.

g) The design criteria should be modified as well. The minimum stability should be 2.25 times, and the range of flow values should be 1.5 times the criteria listed in Table 7.

h) Similar to the normal procedure as shown in Table 6, if the actual specimen thickness varies from 92.5 mm, the correction values as listed in Table 9 should be used to convert the measured stability values to an equivalent value for a specimen with a 92.5-mm thickness.

Table 8 — Stability Corrections for Large Stone Marshall Mixes in 6" Molds

Approximate Height		Specimen Volume	Correlation Ratio
(mm)	(in.)	(cc)	
88.9	3½	1608 to 1626	1.12
90.5	3 9/16	1637 to 1665	1.09
92.1	3 5/8	1666 to 1694	1.06
93.7	3 11/16	1695 to 1723	1.03
95.2	3¾	1724 to 1752	1.00
96.8	3 13/16	1753 to 1781	0.97
98.4	3 7/8	1782 to 1810	0.95
100.0	3 15/16	1811 to 1839	0.92
101.6	4	1840 to 1868	0.90

6.3 Hveem Method of Mix Design

6.3.1 General

The Hveem method of mix design is specifically useful for the design of rut-resistant mixes and it has been extensively used in regions where hot climates and desert areas are prevalent. The Hveem method is applicable to paving mixtures using either asphalt cement or cutback asphalt and containing aggregates up to 25 mm maximum size. The method presented here is applicable to the design of dense-graded HMA paving mixtures.

This method tests the stability of the aggregate structure in a compacted mix specimen. Typically, the optimum binder content determined with the Hveem method is approximately 0.3 percent lower than that determined with the Marshall method. For a given aggregate gradation, one specimen is made for each of four binder contents and tested for various criteria. Using a decision pyramid, the optimum binder content is determined in three steps, selecting each time the highest binder content that meets the criteria.

Test procedures for determining resistance to deformation and cohesion of bituminous mixtures by means of Hveem Apparatus are found in ASTM D1560

6.3.2 Outline of method

The Hveem method begins with the preparation of test specimens. Steps preliminary to specimen preparation are:

a) determining that the proposed materials meet the physical requirements of the project specifications;

b) assuring that the aggregate blend combinations meet the gradation requirements of the project specifications; and

c) having an ample supply of aggregates, which have been dried and sized into fractions.

NOTE These requirements are matters of routine testing, specifications and laboratory technique, which must be considered but are not unique to any particular mix design method.

6.3.3 Approximate asphalt content by the centrifuge kerosene equivalent method

The first step in the Hveem method of mix design is to determine the “approximate” asphalt content. This can be based on past experience or determined by the Centrifuge Kerosene Equivalent (CKE) method. With a calculated surface area and the factors obtained by the CKE method for a particular aggregate or blend of aggregates, the approximate asphalt content is determined by using a series of charts that are presented in this Clause.

6.3.3.1 Equipment

The equipment and materials required for determining the approximate asphalt content (Figure 4) are:

a) small sample splitter for obtaining representative samples of fine aggregate;

b) pans, 114 mm diameter by 25 mm deep;

c) kerosene, 4 liters ;

d) oil, SAE No. 10, lubricating, 4 liters;

e) beakers, 1500 ml;

f) metal funnels, 89 mm top diameter, 114 mm height, 13 mm orifice with piece of 2.00 mm (No. 10) sieve soldered to bottom of opening;

g) timer;

h) centrifuge, manual or power-driven, complete with cups, capable of producing 400 times gravity as seen in Figure 4; and

i) filter papers, 55 mm diameter.

Figure 4 — Apparatus for Hveem CKE Tests

6.3.3.2 Surface area

The gradation of the aggregate or blend of aggregates employed in the mix is used to calculate the surface area of the total aggregate. This calculation consists of multiplying the total percent passing each sieve size (in decimal form) by a “surface-area factor” as set forth in Table 9. Sum these products and the total will represent the equivalent surface area of the sample in terms of square meters per kilogram. It is important to note that all the surface area factors must be used in the calculation. Also, if a different series of sieves is used, different surface area factors are necessary.

NOTE These surface-area factors have been used to calculate an average film thickness using the volume of asphalt binder in the mix. Although this determination of asphalt film thickness can provide a broad, relative indication of mix durability, the Asphalt Institute strongly recommends against comparing this calculated value with specific mix design criteria because of inherent inaccuracies. These surface-area factors do not take into account the specific aggregate shape, but are intended only as an index factor. In addition, in a compacted mixture, some of the asphalt and fine particle mastic is actually shared by adjacent particles rather than each being in an isolated state as assumed.

Table 9 — Surface Area (SA) Factors

Total % Passing Sieve No.	Maximum Size	4.75 mm (No. 4)	2.36 mm (No. 8)	1.18 mm (No. 16)	600 µm (No. 30)	300 µm (No. 50)	150 µm (No. 100)	75 µm (No. 200)
S.A. m^2/kg	.41	.41	.82	1.64	2.87	6.14	12.29	32.77
S.A. (ft^2/lb)	(2)	(2)	(4)	(8)	(14)	(30)	(60)	(160)
• Surface-area factors shown are applicable only when all the above-listed sieves are used in the sieve analysis.								

6.3.3.3 CKE procedure for fine aggregate

The centrifuge kerosene equivalent method involves these steps:

a) Place exactly 100 grams of dry aggregate (representative of the passing 4.75-mm [No. 4] material being used) in a tared centrifuge cup assembly fitted with a screen and a disk of filter paper.

b) Place bottom of centrifuge cup in kerosene until the aggregate becomes saturated.

c) Centrifuge the saturated sample for two minutes at a force of 400 times gravity. (For the manual centrifuge, this force can be developed by turning the handle approximately 45 revolutions per minute.)

d) Weigh sample after centrifuging and determine the amount of kerosene retained as a percent of the dry aggregate weight; this value is called the Centrifuge Kerosene Equivalent (CKE).

Note: Duplicate samples are always prepared in order to balance the centrifuge and to check results. The average of the two CKE values is used unless there is a large discrepancy, in which case the test is rerun.

e) If the specific gravity of the aggregate samples is greater than 2.70 or less than 2.60, make a correction to the CKE value using the formula at the bottom of the chart in Figure 5.

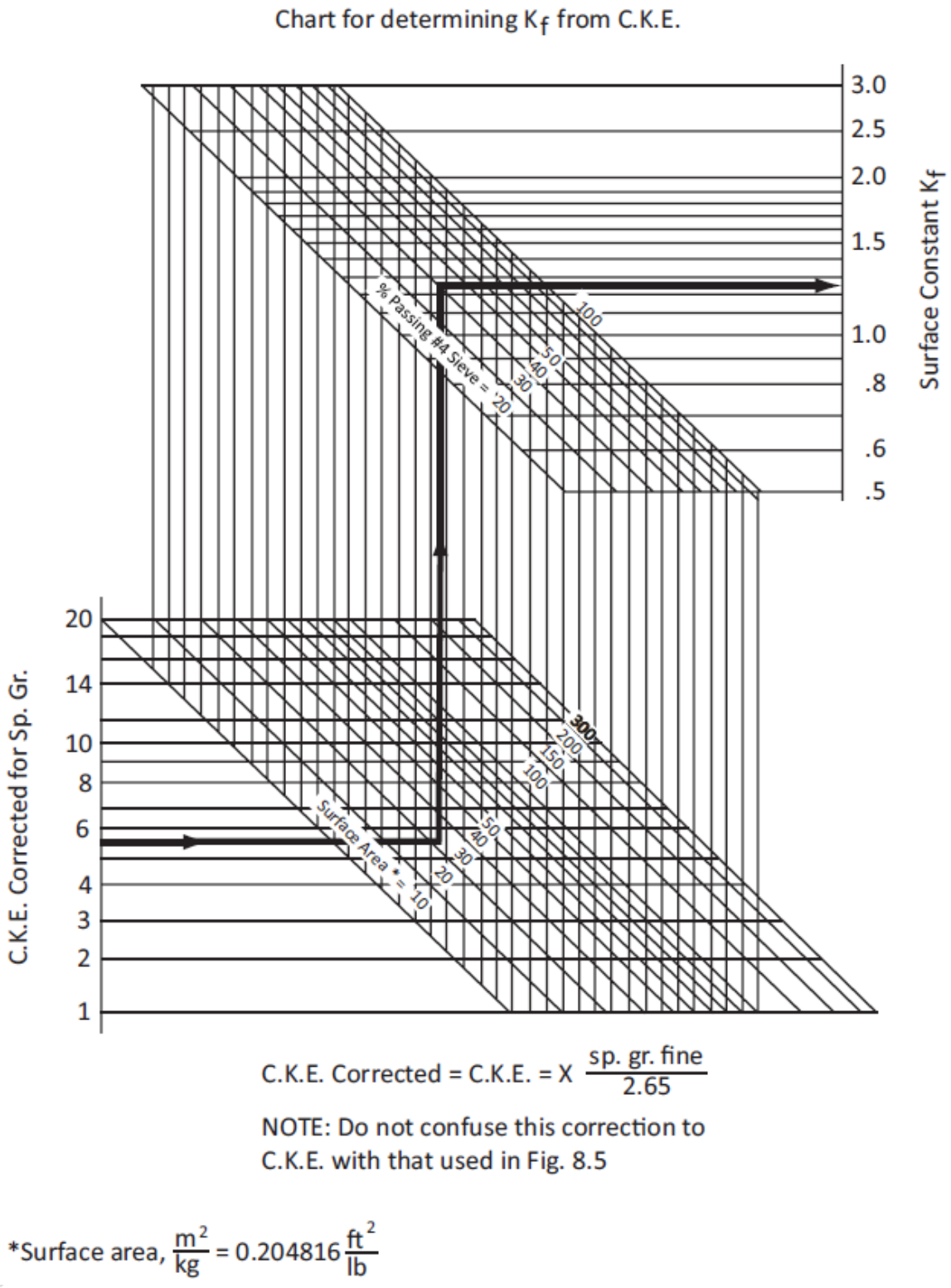

Figure 5 — Chart for Determining Surface Constant for Fine Material, Kf from C.K.E., Hveem Method of Design

6.3.3.4 Surface capacity test for coarse aggregate

The surface capacity (or “oil soak”) test for the larger aggregate involves these steps:

a) Place exactly 100 grams of dry aggregate that passed the 9.5-mm sieve and is retained on the 4.75-mm (No. 4) sieve into a metal funnel (this fraction is considered to be representative of the coarse aggregate in the mix).

b) Immerse sample and funnel into a beaker containing SAE No. 10 lubricating oil at room temperature for 5 minutes.

c) Allow to drain for 2 minutes.

d) Remove funnel and sample from oil and drain for 15 minutes at a temperature of 60°C.

e) Weigh the sample after draining and determine the amount of oil retained as a percent of the dry aggregate weight.

NOTE Duplicate samples are prepared to check results. Average value is used unless there is a large discrepancy, in which case the test is rerun.

f) If the specific gravity of the aggregate is greater than 2.70 or less than 2.60, make a correction to the percent oil retained using the formula at the bottom of the chart in Figure 6.

6.3.3.5 Estimated design asphalt content

These steps are used to make a preliminary estimate of the design asphalt content:

a) Using the CKE value obtained and the chart in Figure 5, determine the value Kf (surface constant for fine material).

b) Using the percent oil retained and the chart in Figure 6, determine the value Kc (surface constant for coarse material).

c) Using the values obtained for K_f and K_c and the chart in Figure 7, determine the value Km (surface constant for fine and coarse aggregate combined). $K_m = K_f$ + correction to K_f. The correction to K_f obtained from Figure 7 is positive if ($K_c - K_f$) is positive and is negative if ($K_c - K_f$) is negative.

d) The next step is to determine the approximate bitumen ratio for the mix based on cutback asphalts of RC-250, MC-250 and SC-250 grades. With values obtained for Km, surface area and average specific gravity, use Case 2 procedures of the chart in Figure 8 to determine the oil ratio.

e) Determine the asphalt content (bitumen ratio) for the mix (see Figure 9) corrected for the grade to be employed, using the surface area of the sample, the grade of asphalt and the oil ratio from Figure 8.

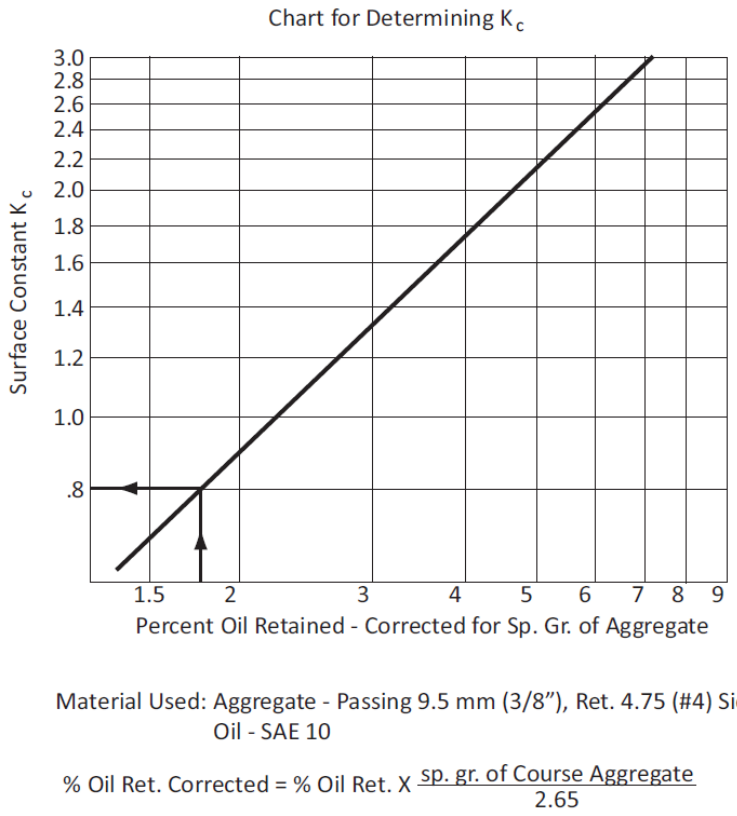

Figure 6 — Chart for Determining Surface Constant for Coarse Material Kc, from Coarse Aggregate Absorption, Hveem Method of Design

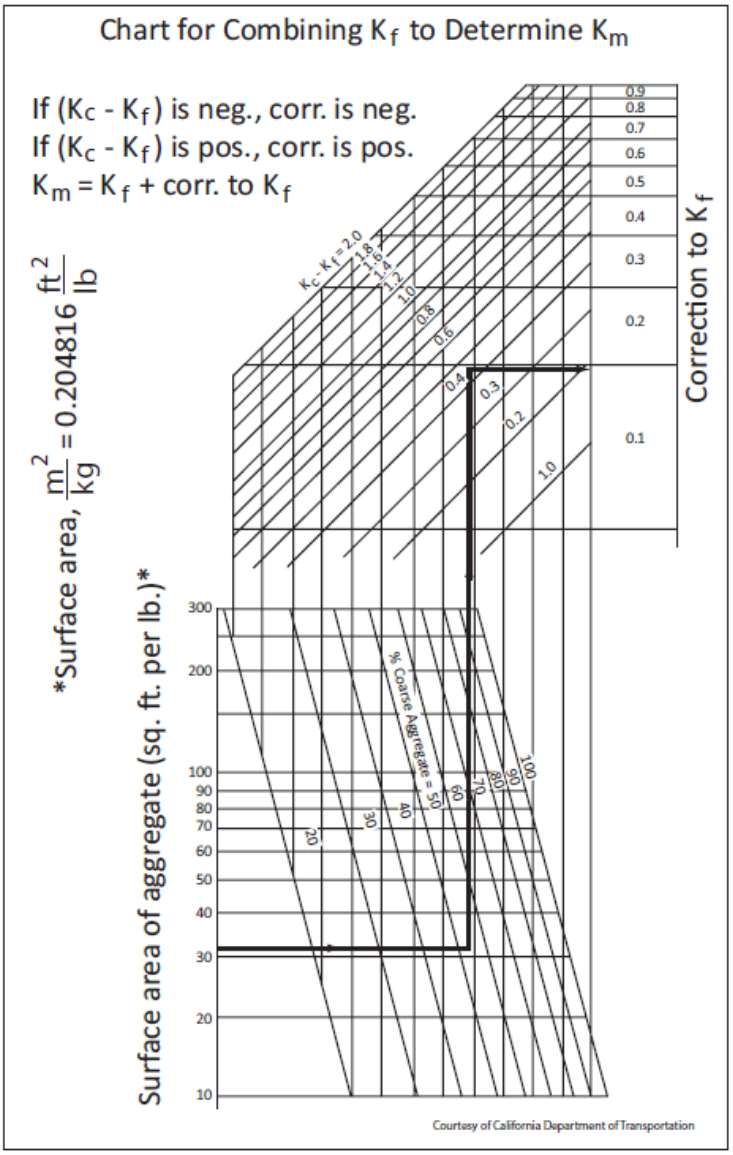

Figure 7 — Chart for Combining K_f and K_c to Determine Surface Constant for Combined Aggregate, K_m, Hveem Method of Design

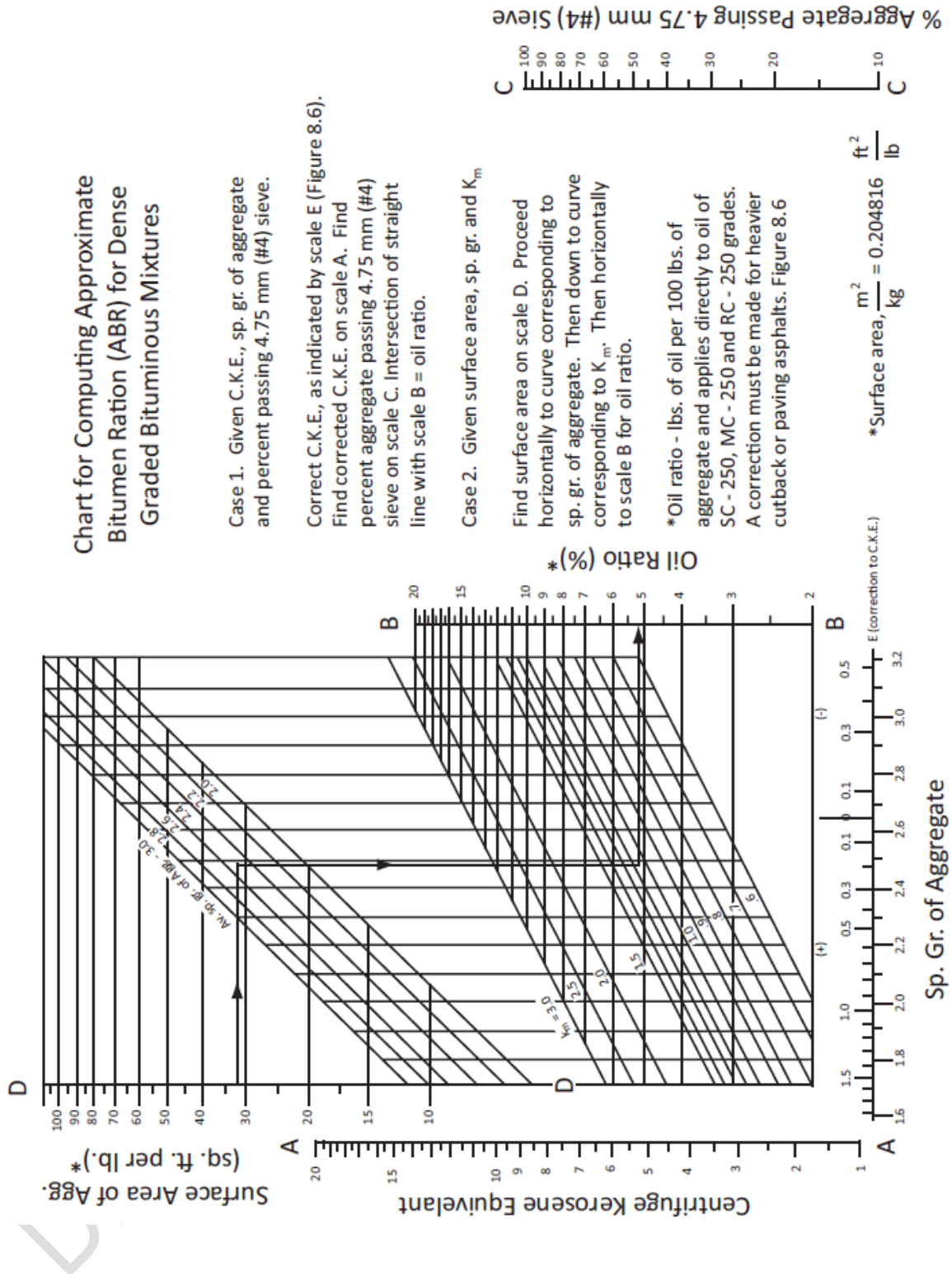

Figure 8 — Chart for Computing Oil Ratio for Dense-Graded Bituminous Mixtures, Hveem Method of Design

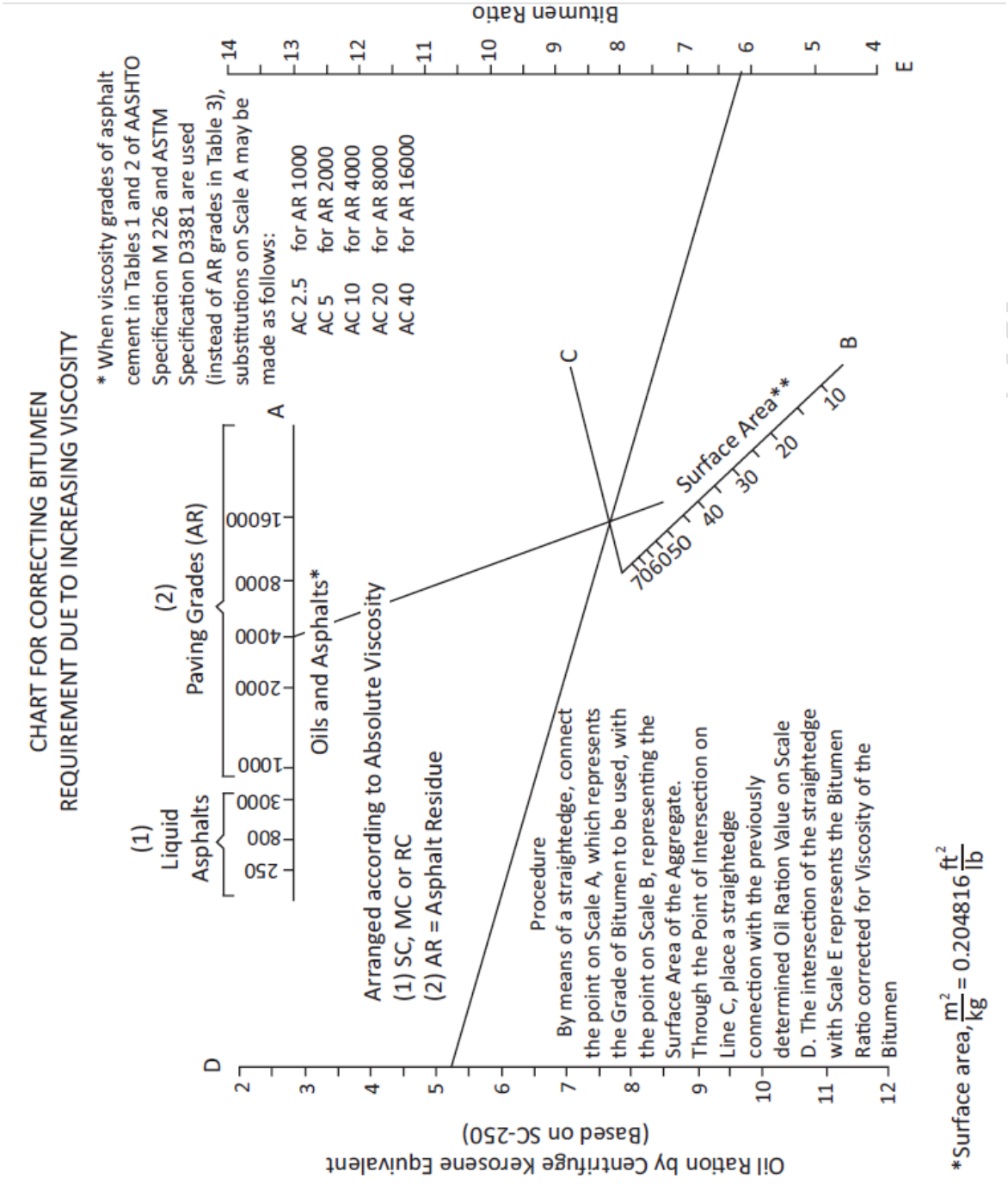

Figure 9 — Chart for Correcting Bitumen Requirement Due to Increasing Viscosity of Asphalt, Hveem Method of Design

6.3.4 Preparation of test specimens

6.3.4.1 Equipment for preparation of Hveem test specimen

The equipment required for the preparation of test specimens is:

a) pans, with a minimum dimension of 250 mm diameter by 50 mm deep, for quartering and mixing fine aggregate, or an equivalent 8 × 12-inch rectangular pan;

b) pans, 200 mm diameter by 45 mm deep, for batching and heating aggregates;

c) pans, 305 mm diameter by 64 mm deep, for mixing aggregate and asphalt;

d) pans, 280 mm by 180 mm by 25 mm for curing mix;

e) large sample splitter for mixing and quartering fine aggregate;

f) electric hot plate, with a surface measuring at least 460 mm by 305 mm, for heating aggregates, asphalt and equipment as required;

g) large oven, thermostatically controlled, capable of 110°C (230°F) temperature, with a tolerance of +/- 5°F;

h) large oven, thermostatically controlled, capable of 60°C (140°F) temperature with a tolerance of +/- 5°F;

i) large oven for drying and preheating aggregates, capable of temperatures up to 165°C (325°F);

j) large scoop for handling hot aggregates;

k) beakers or metal cans, 800 ml, for adding asphalt;

l) thermometer, armored, 35°C (100°F) to 205°C (400°F);

m) balance, minimum 5 kg capacity, sensitive to 0.1 g for weighing aggregates and asphalt;

n) small pointed mixing trowel;

o) large mixing spoon;

p) mechanical mixer (optional);

q) mechanical compactor designed to consolidate the material by a series of individual “kneading action” impressions made by a roving ram having a face shaped as a sector of a 101.6-mm diameter circle. The compactor must be capable of exerting a force of 3.45 MPa beneath the tamper foot. Accessories with the compactor should include two mold holders, an insulated mix feeder trough 460 mm long by 102 mm wide by 64 mm deep, a paddle shaped to fit the trough, and a round-nosed steel rod 9.5 mm diameter by 406 mm long;

r) steel compaction molds, 101.6 mm inside diameter × 127 mm high × 6.4 mm wall thickness;

s) paper disks, heavy paper, 100 mm in diameter, to place in bottom of mold during compaction;

t) hydraulic compression machine, 222 kN capacity;

u) steel shim, 6.4 mm thick by 19.0 mm wide by 63.5 mm long; and

v) gloves, heavy and sturdy, for handling hot equipment.

6.3.4.2 Batch weights

These guidelines are suggested for estimating the aggregate requirements:

a) Compute batch weights for the blend and gradation of aggregates desired.

b) The necessary dry weight of the aggregate for the stabilometer specimens is that which will produce a compacted specimen 63.5 +/- 1.3mm in height. This volume of aggregate will normally weigh about 1,200 grams. To determine the exact batch weight, it is generally desirable to prepare a trial specimen prior to preparing the actual aggregate batches. If the trial specimen height falls outside the limits, the amount of aggregate used for the specimen may be adjusted using:

For International System of Units (SI),

$$Adjusted\ mass\ of\ aggregate = \frac{63.5\ (mass\ of\ aggregate\ used)}{Specimen\ height\ (mm) obtaines}$$

For U.S. Customary Units,

$$Adjusted\ mass\ of\ aggregate = \frac{2.5\ (mass\ of\ aggregate\ used)}{Specimen\ height\ (in.\ ) obtaines}$$

6.3.4.3 Preparation of batch mixes

These steps are provided as a guide in preparing the mixtures for testing:

a) Weigh the various-sized fractions of dry aggregates into suitable pans in accordance with the calculated batch weights.

b) Thoroughly mix each individual batch of aggregate and preheat in oven to desired mixing temperature. Asphalt should be preheated at the same time. The temperature of the aggregate and the asphalt at the time mixing begins is indicated below for the paving grade of asphalt cement being used.

PG grades and modified asphalt binders should be heated as directed in Annex A.

Table 10 — Temperature ranges for preheating aggregates

Grade	Temperature range	
	Minimum	Maximum
AC-2.5, AR-1000 or 200/300 Pen.	99°C (210°F)	121°C (250°F)
AC-5, AR-2000 or 120/150 Pen.	110°C (230°F)	135°C (275°F)
AC-10, AR-4000 or 85/100 Pen	121°C (250°F)	149°C (300°F)
AC-20, AR-8000 or 60/70 Pen	132°C (270°F)	163°C (325°F)
AC-40, AR-1600 or 40/50 Pen	132°C (270°F)	163°C (325°F)

c) When the aggregates and asphalt have reached the desired mixing temperature, form a crater in the aggregates and weigh in asphalt in accordance with the calculated batch weights.

d) Place pan containing aggregates and asphalt for batch mix on hot plate to maintain mixing temperature. Vigorously mix aggregates and asphalt by hand with a pointed trowel or by mechanical mixing until all particles are coated. Take special precaution not to overheat the materials.

e) After mixing is complete, transfer the batch mix to a suitable flat pan and cure for two or three hours at a temperature of 146 ± 3°C (295°F ± 5°F) in an oven equipped with forced draft air circulation. Caltrans uses the CT 304 procedure to cure the mixture for 15 to 18 hours at 60 ± 2.8°C (140°F ± 5°F).

f) After curing is complete, place batch mix in heating oven and reheat mixture to 110°C (230°F). The batch mix is then ready for compaction.

6.3.4.4 Compaction

The compaction of the test specimen is accomplished by means of the Hveem Kneading Compactor, a mechanical compactor that imparts a kneading action type of consolidation by a series of individual impressions made with a ram having a face shaped as a sector of a 101.6-mm diameter circle. With each push of the ram, a pressure of 3.45 MPa is applied, subjecting the specimen to a kneading compression over an area of approximately 2000 square millimeters. Each pressure application is maintained for approximately 0.4 seconds. The detailed compaction procedure follows.

6.3.4.5 Stabilometer specimens

a) Preheat the compaction molds, feeder trough and round-nosed steel rod to approximately the mix compaction temperature.

b) Heat the compactor foot to a temperature that will prevent the mix from adhering to it. The temperature of the compactor foot may be controlled by a variable transformer.

c) Place the compaction mold in the mold holder and insert a 100-mm diameter paper disk to cover the base plate. The steel shim is temporarily placed under the edge of the mold, so the base plate will act as a freefitting plunger during the initial compaction operation.

d) Spread the prepared mixture uniformly on the preheated feeder trough. Using a paddle that fits the shape of the trough, transfer approximately one-half of the mixture to the compaction mold.

e) Rod the portion of the mix in the mold 20 times in the center of the mass and 20 times around the edge with the round-nosed, steel rod. Transfer the remainder of the sample to the mold and repeat the rodding procedure.

f) Place mold assembly into position on the mechanical compactor and apply approximately 20 tamping blows at 1.7 Mpa pressure to achieve a semi-compacted condition of the mix so that it will not be unduly disturbed when the full load is applied. The exact number of tamping blows to accomplish the semicompaction shall be determined by observation.

g) The actual number of tamping blows may vary between 10 and 50, depending upon the type of material, and it may not be possible to accomplish the compaction in the mechanical compactor because of undue movement of the mixture under the compactor foot. In these instances, use a 178 kN static load applied over the total specimen surface by the double-plunger method, in which a free-fitting plunger is placed below and on top of the sample. Apply the load at the rate of 1.3 mm per minute and hold for 30 ± 5 seconds.

h) After the semi-compaction, remove the shim and release mold tightening screw sufficiently to allow free up-and-down movement of mold and about 3 mm side movement of mold.

i) To complete compaction in the mechanical compactor, increase compactor foot pressure to 3.45 Mpa and apply 150 tamping blows.

j) Place the mold and specimen in an oven at 60°C (140°F) for 1 hour, after which a “leveling off” load of 56 kN (12,600 lb.) is applied by the “double-plunger” method head speed = 6 mm/min and released immediately.

NOTE The specimen shall not be pushed to the opposite end of the mold.

6.3.4.6 Swell test specimens

a) Prepare the compaction mold by placing a paraffin-impregnated strip of ordinary wrapping paper 19 mm wide around the inside of the mold 13 mm to 19 mm from the bottom to prevent water from escaping from between the specimen and the mold during the water immersion period of the test. The paper strip is dipped in melted paraffin and applied while hot. Compaction molds are not preheated for swell test specimens.

b) The remainder of the compaction procedure for swell test specimens is the same for the stabilometer test specimens except when compaction is completed in the mechanical compactor, remove mold and specimen from compactor, invert mold and push specimen to the opposite end of mold. Apply a 56 kN static load head speed 6 mm/min. with the “original” top surface supported on the lower platen of the testing press. It is advisable to place a piece of heavy paper under the specimen to prevent damage to the lower platen.

6.3.5 Test procedures

6.3.5.1 General

In the Hveem method, the compacted test specimens are used in these tests and analyses and are normally performed in the order listed:

a) stabilometer test;

b) bulk density determination; and

c) swell test.

The swell test is performed only on specimens prepared for this purpose; the stabilometer and bulk density tests are performed on each of all other test specimens.

6.3.5.2 Equipment for Hveem swell and stability testing

The equipment required for the testing of the 101.6-mm diameter specimens is:

a) bronze disks, perforated, 98.4 mm diameter by 3.2 mm thick, with adjustable stem, for swell measurement (see Figure 11);

b) dial gauge, mounted on tripod, with reading accuracy to 0.025 mm;

c) scale (ml), graduated to read the water volume on top of the specimen, in the 101.6-mm diameter mold at 25 ml intervals, for the purpose of measuring the percolation of water during swell test;

d) round aluminum pans, 190 mm diameter by 64 mm deep;

e) The Hveem stabilometer (see Figure 11) is a triaxial testing device consisting essentially of a cylindrical metal body shell (containing a liquid which registers the horizontal pressure developed by a compacted test specimen as a vertical load is applied), a flexible rubber diaphragm, a pressure gauge, a screw-type hand pump assembly and an air chamber fitted with a needle valve for adjusting the quantity of air in the system. The pump provides a means of obtaining a standard quantity of air in the liquid system of the stabilometer; and

f)scale or other measuring device to accurately determine the height of the compacted test specimen.

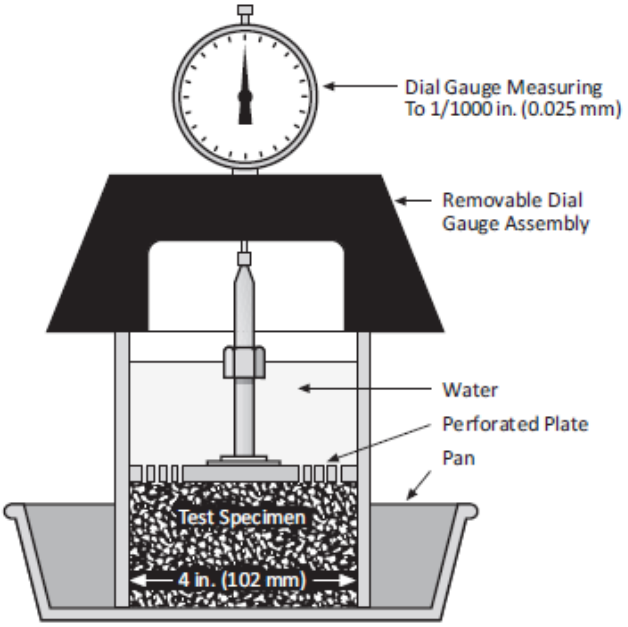

Figure 10 — Swell Test Apparatus

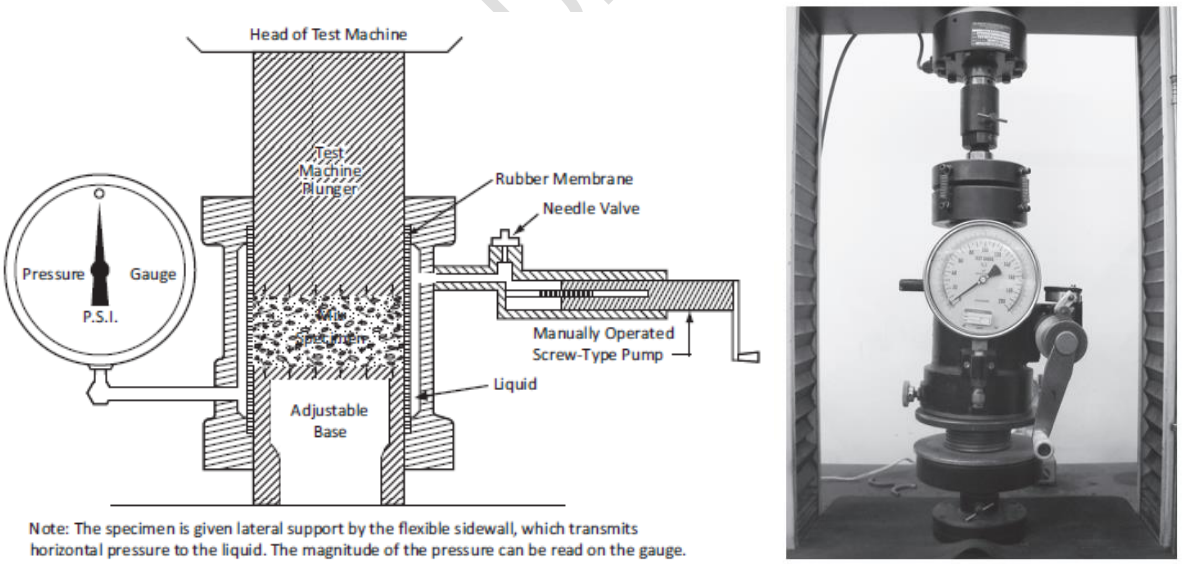

Figure 11 —Diagrammatic Sketch Showing Principal Features of Hveem Stabilometer6.3.5.3 Stabilometer test procedures

The stabilometer measures the lateral pressure transmitted through the specimen from the applied vertical load. The ratio of a given applied unit compressive stress to the transmitted lateral or horizontal pressure is used to determine an index, on a scale ranging from 0 to 100, of the ability of the material under test (at the test temperature) to resist deformation.

NOTE Frequent calibration of the stabilometer should be made during the day as temperature change has considerable effect upon the pressure exerted within the hydraulic system.

These are the steps for measuring the Hveem stability (refer to Figures 11):

a) Place specimens for stabilometer tests (compacted and contained in mold) in oven at 60 ± 3°C for 3 to 4 hours.

b) Adjust compression machine for head speed of 1.3 mm/min. with no load applied.

c) Check displacement of stabilometer with a calibration cylinder and, if necessary, adjust to read 2.00 ± 0.05 turns

d) Adjust the stabilometer base so that the distance from the bottom of the upper tapered ring to the top of the base is 89 mm.

e) Every effort should be made to fabricate test specimens with an overall height between 61 mm and 66 mm; however, if the height is outside of this range, the stabilometer value should be corrected as indicated in Figure 12.

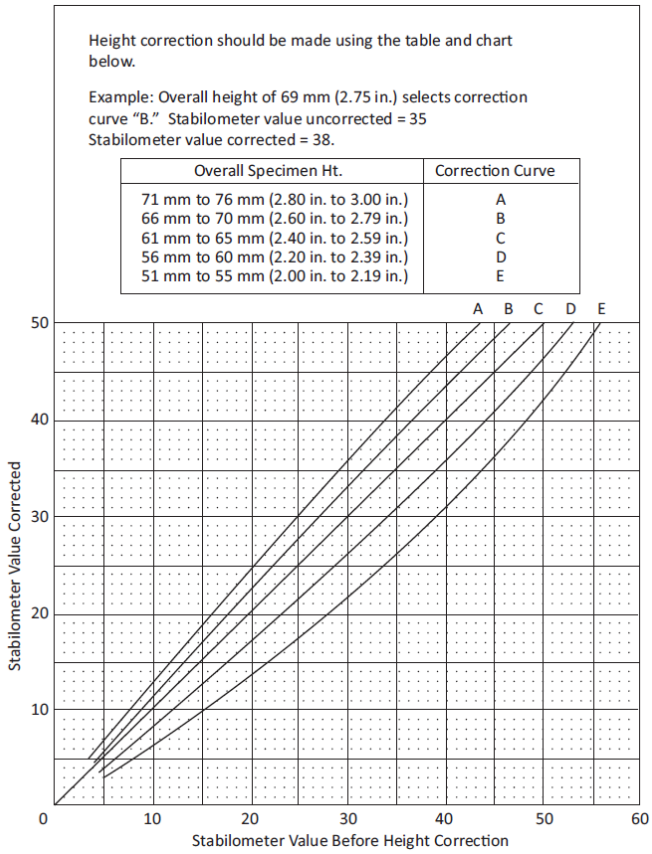

Figure 12 —Chart for Correcting Stabilometer Values to Effective Specimen Height of 64 mm (2.50 in.)

f) Remove the mold with its specimen from the oven and place on top of stabilometer. Using the plunger, hand lever and fulcrum, force the specimen from the mold into the stabilometer.

g) Take care that the specimen goes in straight and is firmly seated. Instead of using the hand lever and fulcrum, this can be done with the compression machine.

h) Place follower on top of specimen and position the entire assembly in compression machine for testing.

i) Using the displacement pump, raise the pressure in the stabilometer system until the test gauge (horizontal pressure) reads exactly 34.5 kPa. (Tap test gauge lightly to assure an accurate reading.)

j) Close displacement pump valve, taking care not to disturb the 34.5 kPa initial pressure. (This step is omitted on stabilometers that are not provided with a displacement pump valve.)

k) Apply test loads with the compression machine using a head speed of 1.3 mm/min. (0.05 in./min.). Record readings of the stabilometer test gauge at vertical test loads of 13.4, 22.3 and 26.7 kN.

Warning: Do not continue loading if the horizontal pressure gauge exceeds 125 psi.

l) Immediately after recording the horizontal pressure reading, under maximum vertical load (26.69 kN, reduce total load on specimen to 4.45 Kn.

m) Open the displacement pump angle valve and by means of the displacement pump adjust test gauge to 34.5 kPa. (This will result in a reduction in the applied vertical load that is normal and no compensation is necessary.)

n) Adjust dial gauge on pump to zero by means of small thumbscrew.

o) Turn the displacement pump handle smoothly and rapidly (two turns per second) and to the right (clockwise) until a pressure of 690 kPa is recorded on the test gauge.

NOTE During this operation, the load registered on the testing press will increase and in some cases exceed the initial 4.45 kN load. This change in load is normal and no adjustment or compensation is required.

Record the exact number of turns required to increase the test gauge reading from 34.5 kPa to 690 kPa as the displacement on specimen (2.5 mm dial reading is equivalent to one-turn displacement).

p) After recording the displacement, first remove the test load and reduce pressure on the test gauge to zero by means of the displacement pump; then reverse the displacement pump an additional three turns and remove specimen from stabilometer chamber.

6.3.5.4 Bulk density determination

The bulk density test is performed on these specimens after the completion of the stabilometer tests as soon as the specimens have cooled to room temperature. The procedure for this test is presented in ASTM D1188, using paraffin- coated Specimens and ASTM D2726, using saturated surface dry specimens.

6.3.5.5 Swell test

These steps outline the swell test procedure.

a) Allow compacted swell test specimen to stand at room temperature for at least one hour. (This is done to permit rebound after compaction.)

b) Place the mold and specimen in 190 mm diameter by 64 mm deep aluminum pan (see Figure10).

c) Place the perforated bronze disk on the specimen, position the tripod with the dial gauge on the mold and set the adjustable stem to give a reading of 2.54 mm on the dial gauge (see Figure 10).

d) Introduce 500 ml of water into the mold on top of the specimen and measure the distance from the top of the mold to the water surface with the graduated scale.

e) After 24 hours, read the dial gauge to the nearest 0.025 mm and record the change as swell. Also, measure the distance from the top of the mold to the water surface with the graduated scale and record the change as permeability or the amount of water in ml that percolated into and/or through the test specimen.

6.3.6 Interpretation of test data

6.3.6.1 Calculations

There are no calculations required for the swell test since the results are reported directly as differences.

The stabilometer measures the lateral pressure resulting from the dilation of the specimen caused by the applied vertical load. The ratio of a given applied unit compressive stress (Pv) to the transmitted lateral or horizontal pressure (Ph) is used to determine the S-value), on a scale ranging from 0 to 100, of the ability of the material to resist deformation. The higher the S-value, the higher the resistance to deformation. The S-values for Hveem compacted asphalt mixture specimens are typically a minimum of 35.

The remainder of the calculations are:

a) Stabilometer value (S); calculate as:

$$S = \frac{22.2}{\frac{P_h D}{P_v - P_h} + 0.222}$$

Where,

S = stabilometer value

D = the number of turns of the air displacement pump handle needed to increase the lateral pressure from 5 psi to 100 psi

P_v vertical pressure (typically 2.76 Mpa [400 psi] = 22.24 kN total load)

P_h the horizontal pressure (stabilometer pressure gauge) reading taken at the instant the vertical pressure Pv is 2.76 MPa or 400 psi (= at 5,000 lbs. vertical load)

b) Density and voids analysis. Using the bulk specific gravity of the test specimens and the maximum specific gravity of the paving mixture determined using ASTM D2041, compute the percent air voids. These values may be plotted as a function of asphalt content as shown in Figure 13, similar to the Marshall procedure, to assist in design.

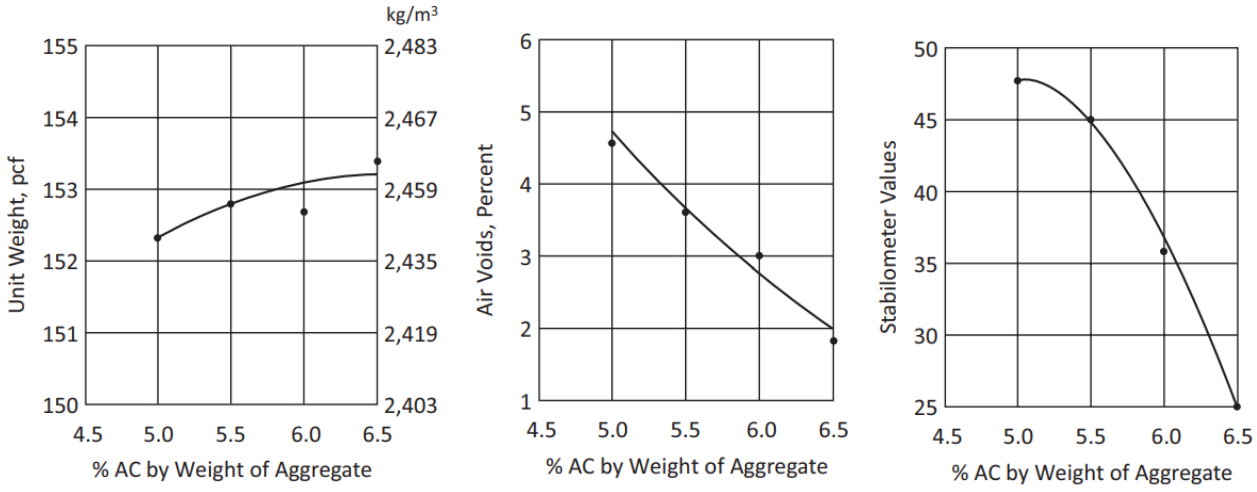

Figure 13 —Test Property Curves for Hot Mix Design Data by the Hveem Method

6.3.7 Design criteria

The suitability of the hot mix design by the traditional Hveem method is determined on the basis of whether the asphalt content and aggregate grading will satisfy the requirements in Table 11: Calculate the air voids content of each specimen.

In applying these requirements, the design asphalt content should be the highest percentage of asphalt the mix will accommodate without reducing stability or void content below minimum values. The design asphalt content is determined from observations of surface flushing or bleeding of specimens after compaction, the stabilometer values and percent air voids.

Table 11 — Hveem mix design criteria

Quality Characteristic	Traffic Category		
	heavy	medium	light
Air voids content (%)	4.0	4.0	4.0
Voids in mineral aggregate (% min.)			
4.75-mm grading	17.0	17.0	17.0
9.5-mm grading	15.0	15.0	15.0
12.5-mm grading	14.0	14.0	14.0
19-mm grading	13.0	13.0	13.0
Stabilometer value (min.)			
12.5mm and 19 mm gradings	37	35	32
Swell	less than 0.762 mm (0.030 in.)		

Notes:

1. Although not a routine part of this design method, an effort is made to provide a minimum percent of air voids of approximately 4 percent.

2. All criteria, and not stability value alone, must be considered in designing an asphalt paving mix.

3. Hot mix asphalt bases that do not meet these criteria when tested at 60°C (140°F) are satisfactory if they meet the criteria when tested at 38°C (100°F) and are placed at 100 mm (4 in.) or more below the surface. This recommendation applies only to regions having a range of climatic conditions similar to those prevailing throughout most of the United States. A different lower test temperature may be considered in regions having more extreme climatic conditions.

4. Traffic classifications:

Light: Traffic conditions resulting in a Design EAL $<10^4$

Medium: Traffic conditions resulting in a Design EAL between 10^4 and 10^6

Heavy: Traffic conditions resulting in a Design EAL $>10^6$

6.3.8 Determination of Optimum Binder Content (OBC)

a) Using Figure 14, insert in Step 1 of the pyramid the asphalt contents used for preparing the series of mix design specimens. Insert the asphalt contents in order of increasing amounts from left to right, with the maximum asphalt content used in the square on the right.

b) Plot percent air voids versus percent asphalt content, and determine by interpolation the percent asphalt content that has 4.0 percent air voids.

c) Select from Step 1 the three highest asphalt contents that do not exhibit moderate or heavy surface flushing and record on Step 2 of the pyramid. Surface flushing and/or bleeding is considered “Slight” (acceptable) if the surface has only a slight sheen. It is considered “Moderate” (unacceptable) if sufficient free asphalt is apparent to cause paper to stick to the surface but no distortion is noted.

d) Surface flushing is considered “Heavy” (unacceptable) if there is sufficient free asphalt to cause surface puddling or specimen distortion after compaction.

e) Select from Step 2 the two highest asphalt contents that provide the specified minimum stabilometer value and enter them in Step 3 of the pyramid.

f) From the plot in (b), determine the theoretical asphalt content that has 4 percent air voids and is within the asphalt range listed in Step 3 of the pyramid. Always stay as close to 4 percent air voids as possible. The selected percent asphalt content is entered in Step 4, and is the optimum binder content (OBC) for the job mix formula (JMF).

g) The asphalt content in Step 4 is the design asphalt content. However, if the maximum asphalt content used in the design set (Step 1) is the asphalt content entered on Step 4 of the pyramid, additional specimens must be prepared with increased asphalt contents in 0.5 percent increments and a new design asphalt content determination should be made.

h) At the final OBC, compact three additional Hveem specimens and determine the final values for Hveem stability and volumetric properties as the average of each of the three test values.

i) Additional mix design voids criteria are contained in clause 5 as well as some factors to consider when selecting the final design asphalt content.

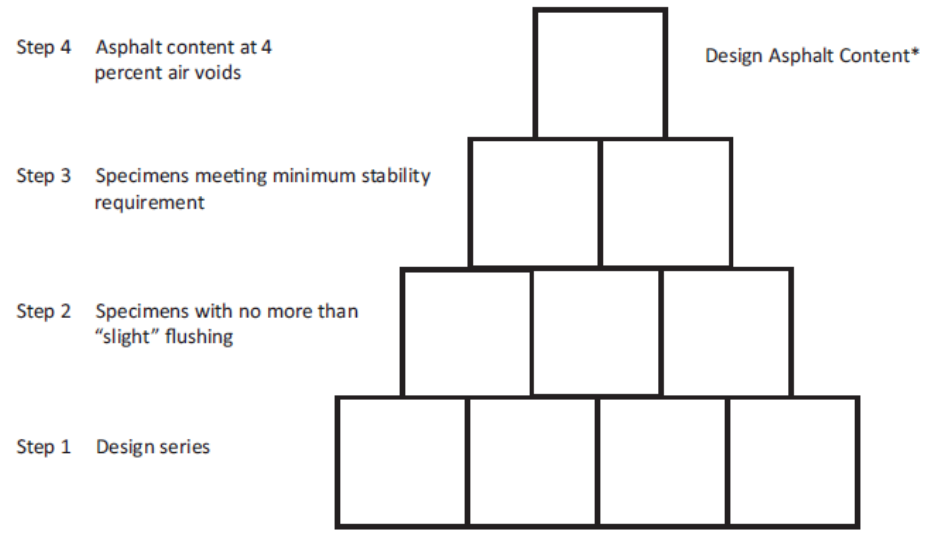

Figure 14 —Procedures for selecting design asphalt content, Heveem Method of Design

7 Recycled Asphalt Pavement (RAP) Materials in the Mix Design Process

7.1 Reclaimed Asphalt Pavement (RAP)

7.1.1 Recycled asphalt pavement (RAP) uses recycled materials that meets all of the specified requirements for mixes produced with the specified requirements for mixes produced with 100 percent virgin materials. Most RAP is produced from milling. It also produced from any existing asphalt pavement by processing it to an appropriate size to be used as a component of a new asphalt mixture. RAP consists of the same components as virgin HMA (aggregate and asphalt binder), it can readily be incorporated into a new mixture.

7.1.2 Guidelines for recycling of bituminous materials are given in Annex B.

7.2 RAP properties

The amount of RAP that is used in the asphalt mixture dictate what properties are needed to be known before the RAP can be used. For all levels of RAP, asphalt binder content and gradation will need to be known. At higher levels of RAP, the physical properties of the asphalt binder will also be needed so that blending charts can be used to select the appropriate grade of virgin, or newly added, asphalt binder.

7.2.1 Determining asphalt binder content and aggregate gradation

Regardless of the amount of RAP that is used in the mixture, the mix design technologist will need to know the asphalt binder content of the RAP so that the total asphalt binder content (RAP binder plus virgin binder) can be properly determined.

The asphalt binder content of RAP is determined using either of the following:

a) **ignition oven procedure** which is detailed in AASHTO T 308, and

b) **solvent extraction** as detailed in AASHTO T 164.

At the conclusion of the procedure, the recovered aggregate should be saved for determining the gradation using AASHTO T 30, “Mechanical Analysis of Extracted Aggregates.”

NOTE Although either procedure can be used, there are disadvantages associated with each. With the ignition oven, degradation is possible with some aggregates, which could lead to a gradation that is finer than the actual gradation of the RAP. The ignition oven also requires a correction factor that may have to be estimated for the RAP and not measured. Experience with local aggregates can indicate if the ignition oven is an appropriate procedure to use. The biggest inconvenience with the second option using the extraction procedure is that it uses solvents.

Both procedures should be capable of producing a sufficient sample size after extraction so that the extracted aggregate can be tested for gradation. Additional extraction or ignition oven runs may be needed to complete

other aggregate property tests as specified. Although either procedure may be used for determining asphalt binder content and aggregate gradation, the solvent extraction procedure should be selected if asphalt binder properties are needed.

7.2.2 RAP aggregate specific gravity

There are two main ways of determining the combined specific gravity of the RAP aggregate.

a) **RAP Specific Gravity Method 1** uses the RAP aggregate that is obtained following a solvent extraction or ignition oven procedure. The aggregate is split into coarse and fine fractions, and individual specific gravity tests are performed. The combined RAP aggregate specific gravity is then determined.

b) **RAP Specific Gravity Method 2**, which uses the RAP as is to determine the maximum theoretical specific gravity, G_{mm}, of the RAP. If the asphalt binder content of the RAP is known (or determined), then the effective specific gravity, G_{se}, of the RAP aggregate can be calculated. This value can then be used as a substitute for the bulk specific gravity, G_{sb}, of the RAP.

• **RAP Specific Gravity Method 2a** uses the same methodology as Method 2 with the exception that an asphalt absorption rate is determined or assumed and the G_{sb} of the RAP aggregate is calculated.

7.2.2.1 Determining RAP G_{mm}

Before performing the G_{mm} test on RAP, it is important that the RAP is properly prepared, as described below:

1. Dry the test sample to constant mass in a forced draft oven at 110 + 5°C (230 + 9°F).

2. Break up the sample similar to a standard G_{mm} sample.

3. Mix the RAP sample thoroughly with a spatula to allow the old RAP binder to coat the uncoated aggregate particles.

Determining the maximum theoretical specific gravity on recycled asphalt or core samples can be confounded by the uncoated faces that result from
To prevent intrusion of water into the sample, add a measured quantity of new binder in the range of 1–3 percent of the total sample weight, as needed to fully coat all particles. Preheat the mix to 60–120°C as you would during a normal mix design, place it in the mixing bowl and add the additional binder to thoroughly coat the mixture and let it cool. When G_{mm} testing is completed, simply back out the added asphalt mass and volume from the original procedure using the following equation when the “Mass Determination in Air” from AASHTO T 209 is utilized:

$$\text{Theoretical Maximum Specific Gravity} = \frac{A - J}{(A + D) - (E + K)}$$

where

A is mass of the oven-dry sample in air, g;

D is mass of the container filled with water at 25°C (77°F), g;

E is mass of the container filled with the sample and water at 25°C (77°F), g;

J is mass of the added asphalt binder in air, g;

K is= volume of the added asphalt binder, cc or

$$\text{ml} = \frac{J}{G_b}$$

where

G_b is the specific gravity of the binder added.

7.2.2.2 Calculating RAP Gse and Gsb

After determining the G_{mm}, the designer can calculate the G_{se} (assuming the asphalt binder content is known) using the equation below:

$$G_{se(RAP)} = \frac{100 - P_{b(RAP)}}{\frac{100}{G_{mm(RAP)}} - \frac{P_{b(RAP)}}{G_{b(RAP)}}}$$

Where

$G_{se(RAP)}$ is effective specific gravity of the RAP

$G_{mm(RAP)}$ is measured maximum theoretical specific gravity of the RAP

$P_{b(RAP)}$ is asphalt binder content of the RAP

$G_{b(RAP)}$ is specific gravity of the RAP asphalt binder

The $G_{mm(RAP)}$ and $P_{b(RAP)}$ are determined from testing. While the $G_{b(RAP)}$ can be determined by testing, it would require that the asphalt binder be extracted and recovered. As such, it is recommended that an assumed $G_{b(RAP)}$ be used. In the absence of other guidance, a $G_{b(RAP)}$ value of 1.04 is recommended.

Theequation for determining Gsb of the RAP aggregate is shown below:

$$G_{sb(RAP)} = \frac{G_{se(RAP)}}{\frac{P_{ba(RAP)} \times G_{se(RAP)}}{100 \times G_{b(RAP)}} + 1}$$

Where

$G_{sb(RAP)}$ is bulk specific gravity of the RAP

$G_{se(RAP)}$ is effective specific gravity of the RAP

$P_{ba(RAP)}$ is asphalt binder absorption of the RAP

$G_{b(RAP)}$ is specific gravity of the RAP asphalt binder

In the preceding equation, the G_{mm} and asphalt binder content of the RAP are used to calculate $G_{se(RAP)}$, as shown earlier. As before, a $G_{b(RAP)}$ value of 1.04 is recommended as an estimated value. Mix design technologists can always measure the $G_{b(RAP)}$ if desired.

7.2.2.3 Consensus aggregate properties

RAP aggregate, like virgin aggregate, must meet the consensus property requirements established in AASHTO M 323. It is important to remember that the consensus property requirements apply to the blend of aggregates, virgin and RAP, and not specifically to the individual aggregates.

To test RAP aggregate individually, it must first be separated into coarse and fine fractions by splitting the sample. Material retained on the 4.75-mm (#4) sieve is used for determining coarse aggregate angularity and flat/elongated particles. Material finer than the 2.36-mm (#8) sieve is used for the fine aggregate angularity test. The sand equivalent test is not conducted on RAP aggregate since it has already been coated with asphalt binder and the clay content, if any, will have been removed during the extraction or ignition oven procedure.

7.2.2.4 RAP asphalt binder properties

At higher levels of RAP, it may be necessary to know the properties of the RAP asphalt binder so that an appropriate virgin asphalt binder grade can be selected. If RAP binder properties are needed, then a solvent extraction/recovery procedure should be performed on a sample of the RAP. Solvent extraction and recovery of asphalt binder can be performed following the procedures in AASHTO T 164, followed immediately by binder recovery following the procedures in ASTM D5404.

Once the asphalt binder has been recovered, it should be tested using the procedures in AASHTO M 320 to determine the critical temperatures (sometimes called the “true grade” or “continuous grade” temperatures) where the specification criteria is exactly met. Determination of critical temperature requires testing at two or more temperatures, preferably bracketing the specification value.

7.2.2.4.1 Using RAP asphalt binder properties to select an appropriate virgin binder grade

The following guidelines given in Table 12 are recommended for selection of RAP mixture

Table 12 — Binder Selection Guideline for RAP Mixtures

Recommended Virgin Asphalt Binder Grade	RAP Percentage
No change in binder selection	< 15
Select virgin binder one grade softer than normal (e.g., select a PG 58-28 if a PG 64-22 would normally be used)	15 to 25
Follow recommendations from blending charts	> 25

NOTE Selection of an asphalt binder grade to use in a mixture is usually left up to the project designers. The selected binder grade is based on a number of factors such as climate and anticipated traffic loading at the project site. This is referred to as the project binder grade. Because of the time and testing expense of determining RAP binder properties for blending, many asphalt mixture designers elect to use a percentage of RAP that will not require binder properties and blending charts and/or equations.

7.4 Developing the mix design

7.4.1 Determining combined aggregate gradation

RAP should be treated like any other aggregate when determining the combined aggregate gradation—except that the mix designer must remember that the RAP contains aggregate and asphalt binder. Unlike virgin

aggregates, the percentage of aggregate in the RAP will always be less than the total percentage of RAP added to the mixture.

To determine the combined gradation, the RAP_{Blend} percentage (i.e., how much RAP total will be used) is multiplied by the RAP aggregate percentage to get a stockpile percentage for determining combined mixture gradation. The equation is shown below:

$$RAP_{stockpile} = RAP_{Blend} \times \left(1 - \frac{P_{b,RAP}}{100}\right)$$

where

$RAP_{stockpile}$ the stockpile percentage of the RAP used in aggregate blending calculations

RAP_{Blend} the total amount of RAP used in the mixture, in percent

$P_{b,RAP}$ asphalt binder content of the RAP, expressed as a percent

The next step is to adjust the stockpile percentages of the virgin aggregates. This is done by first dividing the amount of each virgin aggregate used by the total amount of virgin aggregate in the mix and then proportioning that percentage as a function of the difference between the RAP stockpile and RAP_{Blend}. The equation is as follows:

$$VirginAgg_{stockpile} = VirginAgg_{Blend,n} + \frac{VirginAgg_{Blend}}{\left(\sum_1^n VirginAgg_{Blend,n}\right)} \times (RAP_{Blend} - RAP_{stockpile})$$

where

$VirginAgg_{stockpile}$ is the stockpile percentage of a virgin aggregate used in blending calculations

$VirginAgg_{Blend,n}$ is the amount of a virgin aggregate (n) used in the mixture, in percent (where n = 1 is Stockpile #1, n = 2 is Stockpile #2, etc.)

RAP_{Blend} is the total amount of RAP used in the mixture, in percent

$RAP_{stockpile}$ is the stockpile percentage of the RAP used in aggregate blending calculations

7.4.2 RAP batching and handling in the lab

When preparing to conduct a mix design, the virgin aggregates are batched according to their stockpile percentages and blended into a combined batch that is then preheated at the mixing temperature (actually slightly above mixing temperature, as discussed earlier, to account for heat loss during weighing) prior to beginning the mix design process. When using RAP, the RAP aggregate and asphalt binder are batched together and must be kept separate and not heated like the virgin aggregates.

7.4.3 Heating of RAP and virgin aggregates in the mix design process

Normally, virgin aggregates are batched, combined and placed in an oven overnight at the appropriate mixing temperature. Unfortunately, since RAP contains asphalt binder, it cannot be heated in the same manner without affecting its properties.

For RAP mixtures, it is recommended that the RAP be batched separately in no more than 1- to 2-kilogram (1,000 to 2,000 gram) batches and heated for no more than two hours at 110°C (230°F). This is a sufficient

time to heat the RAP and remove any extraneous surface moisture. Higher temperatures and longer heating times have been shown to change the properties of some RAP materials.

To compensate for the introduction of a lower temperature material, the virgin aggregate should be heated above the mixing temperature by a certain amount. Although actual mix temperatures vary, a good rule of thumb is to increase the temperature of the virgin aggregates by 0.5°C (0.9°F) for every percent of RAP used in the mix (RAPBlend). Thus for a mixture with RAPBlend = 25 percent, the mixing temperature of the virgin aggregates should be raised by 12.5°C (22.5°F).

The mixing temperature of the virgin asphalt binder should not be adjusted. To mix each sample, the pre-batched virgin aggregate is added to a mixing bowl (or bucket mixer) and the heated RAP is added and quickly mixed with the virgin aggregates. A crater is formed just as in the normal mixing process, and the virgin asphalt binder is added in the appropriate amount and the mixing process is started.

7.4.4 Determining new virgin binder quantity

Because the pre-batched RAP sample contains asphalt binder in addition to aggregate, the weight of the RAP binder must be accounted for so that the total asphalt binder content is correct. This is done by calculating the total weight of asphalt binder to be used for a specified asphalt binder content and then subtracting the RAP binder to get the weight of the virgin asphalt binder to be added. This is shown in the equation below:

$$\text{Virgin Binder Weight} = \text{Total Agg Weight} \times \frac{P_b}{P_s} - (\text{RAP Batch Weight} - \text{RAP Agg Weight})$$

where

Virgin Binder Weight the weight of the virgin binder to be added

Total Agg Weight is the total weight of virgin and RAP aggregates

P_b desired asphalt binder content, percent

P_s aggregate content, percent ($100 - P_b$)

RAP Batch Weight total weight of RAP that was batched

RAP Aggregate Weightweight of RAP aggregate in the blend

Annex A
(normative)

Laboratory Mixture Testing

A.1 General

Laboratory asphalt mixture testing is primarily conducted to determine two fundamental asphalt mix properties the bulk specific gravity of the mixture (G_{mb}) and the theoretical maximum specific gravity of the mix (G_{mm}). These values are utilized in calculating volumetric properties discussed in (clause 5) which are specified requirements for most asphalt mix design procedures. Laboratory asphalt mix specimens are also prepared for further mixture testing such as moisture sensitivity or other recommended or specified performance tests. The specific gravity of aggregates, binders and asphalt mixtures are determined using standardized procedures.

A.2 Selection of trial binder contents, compaction temperatures and mixing times

A.2.1 Selecting the trial binder content

Most asphalt mixture design procedures are conducted by preparing multiple trial mixtures at different binder content percentages. Decisions to be made regarding what binder contents to use in the mix design process include the number of trial binder contents, the increments between binder contents and the range of binder contents to be covered by the trial blends. The most commonly selected increment between binder percentage points is 0.5 percent. However, if the designer is unfamiliar with aggregates being used, or if other materials or additives are used such as RAP, RAS, lime, WMA, etc., more points at smaller increments over a wider range may be needed.

The ultimate goal is to prepare batches that bracket the anticipated design binder content. The designer should evaluate the aggregate type, the aggregate structure and any additional materials or additives before selecting the beginning and ending binder points. There are several guidelines to keep in mind:

a) binder demand increases as the nominal maximum aggregate size of the mix decreases;

b) absorptive aggregates have a greater binder demand;

c) for a given nominal maximum aggregate size, a fine aggregate gradation will require more binder than a coarse aggregate gradation;

d) if higher VMA is anticipated due to hard, angular aggregates, more binder will be required;

e) mixes with a higher P200 tend to require more binder than those with a lower P200; and

f) ask the manufacturer for guidance when proprietary additives are used.

In the end, the designer should never extrapolate a higher or lower design binder content from outside the range of trial points.

A.2.2 Mixing and compaction temperatures

For years, asphalt mix design procedures have used equiviscous temperature ranges for selecting laboratory mixing and compaction temperatures. The purpose of using equiviscous mixing and compaction temperatures in laboratory mix design procedures is to normalize the effect of asphalt binder stiffness on mixture volumetric

properties. By using this procedure, a particular asphalt mixture of the same aggregate structure will exhibit very similar volumetric properties regardless of whether a hard or soft asphalt binder is used.

In the equiviscous method, the viscosity of the asphalt binder is determined at two test temperatures, establishing a relationship between temperature and viscosity as shown in Figure A.1. Compaction temperatures are determined where the viscosity-temperature line crosses the compaction viscosity range of 0.28 ± 0.03 Pa-s. Mixing temperatures are determined where the viscosity-temperature line crosses the mixing viscosity range of 0.17 ± 0.02 Pa-s.

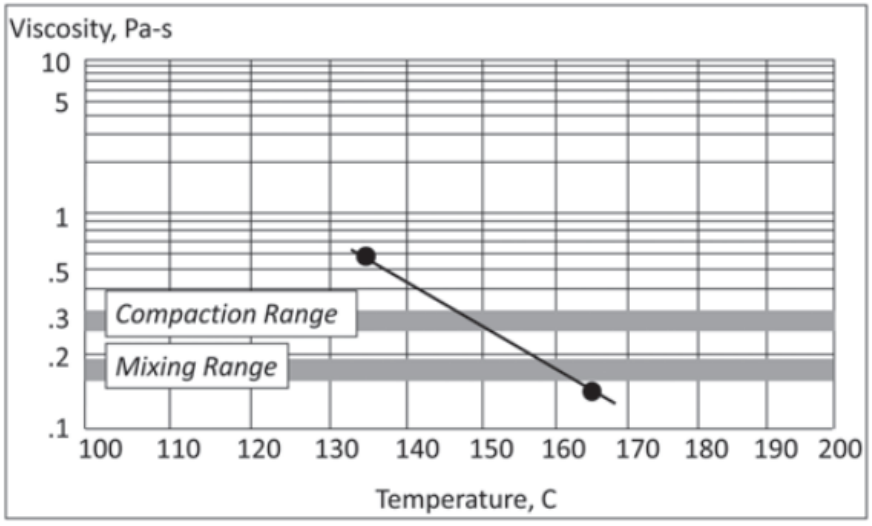

Figure A.1 — Mixing and compaction temperature chart

A.2.3 Laboratory mixing operations

The goal of laboratory mixing operations is to produce uniform batches of properly coated HMA mixtures. Mixing is typically done with either a planetary mixer with wire whips or a five-gallon bucket mixer (see Figure A.2). The following points are important to remember in the mixing operation:

a) Place the aggregate and binder in the oven at the mixing temperature for at least two hours before mixing. To avoid excessive aging of the binder, do not allow it to stay at the mixing temperature for much over the time needed to bring it to temperature and complete the mixing operation.

b) Place all mixing bowls, whips and molds in the oven 30 minutes to an hour before mixing. Keep enough molds in the oven to rotate their use, always keeping a hot mold available.

c) Keep the binder in smaller containers (no more than a gallon) to avoid aging it from constant reheating. Transfer the hot binder to quart cans or other small containers as needed to make it safer and easier to pour. Small stainless steel pitchers from commercial restaurant supply stores work very well.

d) Place the spatula blade on a hot plate, making sure that the wooden handle does not touch the hot plate (usually a heavier nonflammable object is placed on the blade to keep the spatula from slipping off).

e) Keep a stack of paper squares next to the scale to intercept the poured binder stream when it is about to reach the proper weight and to dip out excess binder.

f) It is good practice to place something on the scale to protect the electronics from overheating when the hot bowl/bucket is placed on it.

g) “Butter” the mixing bowl and whip by mixing a dummy batch before subsequent design batches to coat the equipment with binder as an aid in maintaining consistent binder contents.

h) Make sure to have an oven set at the compaction temperature ready to receive the freshly mixed batches.

Figure A.2 — Mixing machines (Five-gallon bucket mixer on left side and Weighing binder into mixing bowl on right side)

When you are ready to mix, place the mixing bowl/bucket on the scale and tare it. Pour the heated aggregate batch into the bowl and verify the required weight. Form a crater in the center of the aggregate to receive the binder and keep it from flowing to the edges of the bowl. Add the correct number of grams of binder, dipping out any excess binder with a folded paper dipper. Mix until all of the aggregate is thoroughly coated.

It is suggested that the mixing should be for approximately 60 seconds for single specimen batches and approximately 120 seconds for multiple specimen batches.

When transferring the mixture from the mixing bowl to the conditioning pans, make sure to thoroughly scrape the mixed fines and asphalt from the bowl and whips into the conditioning pans with a hot spatula.

A.2.4 Mixture conditioning

It is recommended that all G_{mb} and G_{mm} mixture samples be conditioned a minimum of 2 hours prior to the compaction of G_{mb} samples and the cooling and testing of G_{mm} samples, regardless of the mix design procedure utilized. Aggregate sources that have high water absorption values (above 2.0 percent) should be conditioned for an extended period of time (up to 4 hours).

A.2.4.1 Equipment

The equipment required for conditioning the mixture includes:

a) oven—a forced-draft oven capable of maintaining the desired temperature setting within ± 5.4°F (3°C);

b) **thermometers**—having a range of 122°F (50°C) to 500°F (260°C), readable to 1.8°F (1°C); and

c) **miscellaneous**—a shallow metal pan for heating uncompacted asphalt mix, a metal spoon or spatula, tim

A.2.4.2 Procedure

a) Place the mixture in a shallow metal pan and spread it to an even thickness between 25 and 50 millimeters in depth. Place the mixture and pan, in the oven for 2 hours ± 5 minutes at a temperature equal to the mixture's compaction temperature ± 5.4°F (3°C).

b) Note that the conditioning time may need to be increased to be more representative of field conditions when higherabsorptive aggregates (more than 2 percent) are used, subject to agency approval.

c) Stir the loose mixture every 60 ± 5 minutes to maintain uniform conditioning.

d) Remove the mixture and pan from the oven after 2 hours ± 5 minutes. The mixture is now conditioned for further testing. The procedure for short-term conditioning for mixture mechanical property testing is similar to that for volumetric mix design, but the conditioning time is 4 hours ± 5 minutes, and the oven temperature, 275°F (135°C) ± 5.4°F (3°C).

A.3 Laboratory compaction

A.3.1 Generaal

The laboratory compaction effort is intended to replicate the ultimate or final compacted condition of the pavement after being exposed to several years of traffic loading. The designer should understand that different compaction methods affect the aggregate orientation and density profiles within the specimen. Therefore, different types (not meaning different models within the same type) of compaction methods should not be used for mix comparison. The goal of laboratory compaction is to both simulate the compaction expected in the field and ensure sufficient repeatability for acceptable consistency between different labs and technicians. The steps preliminary to specimen preparation are:

a) ensure all materials used in the design process have been sampled according to an accepted procedure and are representative of the materials to be used on the project;

b) ensure all materials proposed for use meet the physical requirements of the project specifications;

c) determine the appropriate mixing and compaction temperatures, either through the use of a viscosity versus temperature chart or by local agency specification; and

d) determine the bulk specific gravity of all aggregates used in the blend and the specific gravity of the asphalt binder for performing density and voids analyses.

These requirements are matters of routine testing, specifications and laboratory technique that must be considered for any mix design method.

This following are the three most common compaction methods for volumetric mix design and quality control testing:

• Gyratory—Superpave;

• Impact—Marshall; and

• Kneading—Hveem.

Detailed discussion of these compactors and their use can be found in their respective design methodology in Clause 6.

A.4 Determining bulk specific gravity, G_{mb}

Determining the bulk specific gravity (G_{mb}) of lab-molded specimens and roadway cores is a fundamental component of asphalt mix design and testing. All specific gravity computations involve a mass divided by a volume multiplied by the unit mass of water. In this case, the mass includes both the mass of the aggregate and the mass of the binder. The volume includes the effective volume of the aggregate, the volume of the binder and the volume of the air voids within a compacted specimen.

There are three main standardized methods for determining the bulk specific gravity in the laboratory. The most common is outlined in ASTM D2726, is intended for relatively nonabsorbent (impermeable), densegraded specimens and uses the saturated surface dry (SSD) mass in the calculation.

There other two methods are intended for relatively absorbent (permeable) specimens. The first is the CoreLok method, which is outlined in ASTM D6752 and the paraffin-coated method described in ASTM D1188 intended to be used for compacted mixture specimens with water absorption (infiltration) greater than 2.0 percent by volume.

A.4.1 Determining theoretical maximum specific gravity, G_{mm}

Determining the theoretical maximum specific gravity (G_{mm}) of loose asphalt mixtures is another fundamental component of asphalt mix design and testing that involves a mass divided by a volume multiplied by the unit mass of water. In this case, the mass includes both the mass of the aggregate and the mass of the binder. The volume includes only the effective volume of the aggregate and the volume of the binder. If G_{mb} and G_{mm} samples had the same dry weight in air, the numerators of the specific gravity equation would be the same for G_{mb} and G_{mm}, but the denominator of the G_{mm} calculation is smaller because it does not include the volume of air. Therefore, G_{mm} must always be a larger number than G_{mb}. Theoretically, if a G_{mb} sample could be compacted until 0 percent air voids remain, the G_{mb} and G_{mm} would be equal.

The most commonly used practice for determining the theoretical maximum specific gravity is oulined in ASTM D2041.

There are basic steps in determining the theoretical maximum specific gravity are as follow:

e) The loose mix is warmed and separated into loose, individually coated aggregates.

f) A minimum mass, of the dry loose mix is split out and placed in a metal bowl or calibrated pycnometer and covered with water.

g) A vacuum lid is fitted and secured to the bowl or pycnometer and placed on a vibratory shaker table. A vacuum pump is started and the manometer or absolute pressure gauge reading is used to determine the proper vacuum adjustment. Once the proper (almost absolute, 27.5 mm Hg) vacuum is obtained, the shaker table is started. This provides gentle agitation to help in the removal of any air between particles. The agitation ensures that the air in the mixture is as close as possible to zero.

h) The theoretical maximum specific gravity is calculated using the equation for the specific procedure utilized. Gmm is the mass of the coated aggregate divided by the volume of coated aggregate. Air voids are calculated from the bulk and maximum specific gravities (Gmb and Gmm). The ratio of these two specific gravities is actually the percent by volume of solids (in decimal form).

NOTE A common source of error with this test is that technicians do not calibrate (verify) the mass of the vacuum container filled with water often enough. This is not usually a problem in labs where only distilled water is used for the test, but field labs often have water tanks that serve the entire lab and are refilled periodically, sometimes from different sources. Because it only takes a few minutes to calibrate, more consistent results will be generated if the vacuum containers are calibrated daily or even before each test.

A.5 Effect of binder content G_{mb} and G_{mm}

The effects of asphalt binder content (P_b) on G_{mb} and G_{mm} are illustrated in Figure A.3. It is important to remember that G_{mb} is measured on a compacted mixture sample. As P_b increases, more lubricity is added to the mixture which allows the specimen to compact and slightly reduce the volume, while at the same time the mass of the specimen is also increasing as the binder fills the voids within the compacted aggregate structure. The slight reduction in volume in combination with the increasing mass causes the specific gravity (density) of the compacted sample to increase. As the voids become filled with binder, the volume of the sample begins to increase. This increasing volume is due entirely to the additional binder being added which begins to reduce the overall specific gravity of the compacted specimen.

The effects of increasing P_b on G_{mm} are quite different. As the P_b increases the percent stone (P_s) decreases. Since there is no compaction or air voids involved with the measurement of G_{mm}, the volume of a G_{mm} sample always increases because the volume of binder being added is roughly 2.5 times the volume of stone that is being removed from the mixture. This makes the G_{mm} property very sensitive to binder content. This also shows the importance of obtaining representative samples of mix when conducting G_{mm} testing. If a sample is segregated and is too coarse, the P_b will be artificially low, resulting in a G_{mm} value that is too high. If the segregated sample is too fine compared to the mixture being produced, the binder content of the material tested will be high and the resulting G_{mm} test result too low.

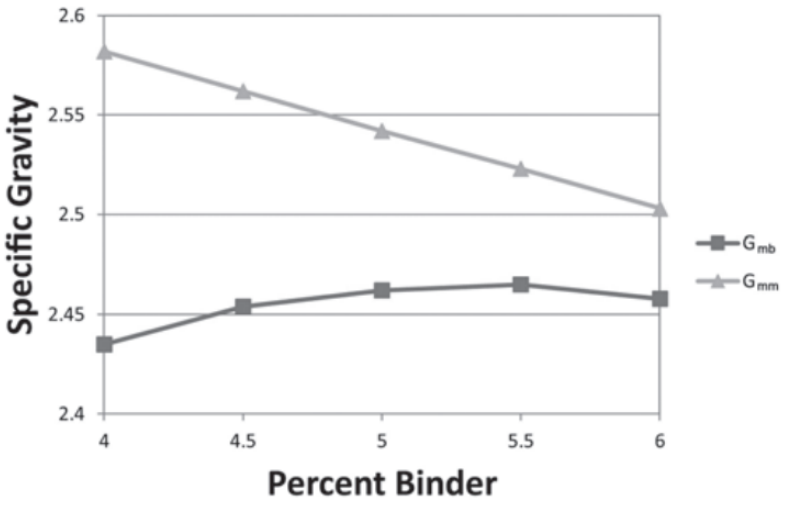

Figure A.3 — G_{mm} and G_{mb} relationship to binder content

Annex B
(normative)

Recycling of bituminous materials

B.1 General

The use of thick bituminous surfacings in developing countries is increasing as traffic loads increase. When they become worn out the recycling of such materials reduce costs and damage to the environment by reducing the exploitation of local natural resources. The types of asphalt that are most likely to be available for recycling are those that have become severely age-hardened and cracked or surfacings that have failed through plastic deformation. Aged asphalt can be expected to be brittle and to contain very hard bitumen. In contrast, asphalt that has deformed plastically is likely to contain bitumen that has suffered very little age hardening. These two types of material present different problems for recovery, stockpiling and re-use.

It is therefore important to determine the variations in properties of the bitumen in RAP and how this will be taken into account in the mix design process.

B.2 Methods of recycling

B.2.1 Reclaimed Asphalt Pavement (RAP), or millings, are crushed and used as granular materials for fill or lower pavement layers or re-used in a bituminous material, either by cold mix or hot mix recycling. These recycling processes can be carried out either in-place or at a central plant. The greatest cost savings will be obtained when RAP is used to produce good quality bitumen- bound material and it's use as unbound material should be regarded as the minimum target for recycling.

B.2.2 The decision to recycle asphalt, its appropriate use and the quality that can be achieved, will be determined by a number of factors which include the following:

a) availability of suitable recycling plant;

b) the thickness of the existing bituminous layer;

c) the effect on traffic management

d) the level of quality control that can be achieved in the recycling process; and

e) the variability in the properties of the existing material.

B.2.3 Standard 'cutting-out' and crushing equipment can be very effective for producing well graded RAP from brittle age-hardened asphalt. This material, modified if necessary by the addition of fresh aggregate, may be suitable for use in any pavement layer. However, it is recommended that RAP is not used to manufacture bituminous wearing courses unless it can be demonstrated that the high degree of uniformity and the close tolerances required for this critical layer can be achieved.

B.2.4 The wider tolerances allowed for bituminous road base and, to a lesser extent, for binder courses, make these mixes more suitable for incorporating RAP. The uniformity and quality of the RAP and the type of recycling plant will determine the percentage of RAP that can be used in the mixes. Typically, this will range from 20 to 50 %.

B.2.5 When RAP is to be used in a pavement layer, good quality control of the RAP stockpiles will be vital to the manufacture of consistent HMA. This may require a considerable amount of testing. The presence of old multiple surface dressings may be acceptable if they have age-hardened. However, where there has been

heavy patching or the quality of the seal is variable, recycling may be limited to in situ pulverisation and stabilisation.

B.2.6 Powerful pulverisers are available which makes it possible to carry out cold in situ recycling with fresh materials being incorporated as necessary to produce a layer of the required quality.

B.3 Suggested method of sampling existing asphalt

B.3.1 A feasibility study will be necessary to assess the variability of the existing material, to establish that a suitable mix design can be achieved, and that it can be manufactured with the available plant. During the feasibility study, samples must be cut from the existing asphalt for analysis. A balance must be found between costly and time consuming testing and the need for sufficient samples to determine material variability. The sampling pattern should take account of visually obvious variability such as:

a) contaminated ‘oil lanes’;

b) wheelpaths that look ‘rich’ in bitumen, indicating a dense material in which bitumen hardening may not be as severe as elsewhere in the pavement;

c) material which looks rich in bitumen and may have deformed plastically; and

d) cracking or fretting indicating that appreciable bitumen hardening has occurred.

B.3.2 Identification of road lengths with apparently uniform appearance will help to establish short representative sections which can be tested. Based on these sections a suitable pattern of testing can be established. The intention should be to stockpile separately severely age-hardened materials, typically with penetration values of less than 20, from less hardened materials and to discard badly contaminated material from ‘oil lanes’. Detailed assessment of stockpile management should be finalised after a desk study has been carried out to show how the various RAP materials can be combined with fresh aggregate to produce acceptable mixes. The following tests should be carried out to determine material properties:

a) particle size distribution;

b) bitumen content;

c) viscosity of recovered bitumen; and

d) an assessment of crushability.

B.4 Methods of obtaining RAP

B.4.1 General

RAP can be obtained by milling or it can be cut from the road in lumps which must be crushed. An assessment of the likelihood of obtaining a well crushed material with the available plant must be made, preferably at the feasibility stage. Milling is particularly useful where traffic access must be retained during the removal of damaged asphalt. Either method is suitable when the road is closed to traffic during the rehabilitation work.

B.4.2 Asphalt millings

Asphalt millings are obtained by planing, in a layer by layer fashion, using a mobile plant and are typically consistent in their lump-size distribution. They can normally be used as granular material, as won, or with minimum screening to remove any over-size material.

B.4.3 Crushed asphalt

Crushed asphalt is commonly obtained by using horizontal impact crushers or hammermill impact crushers. Jaw/roll combination crushers are not suitable for processing RAP which contains 'soft' bitumen because 'pancaking' can occur on warm days and the material will remain agglomerated.

B.4.4 Granulated asphalt

Granulated asphalt is produced in a specialised plant, known as a granulator, or in milling/grinding units. These units are not crushers and are designed only to break the bitumen-asphalt bond.

B.5 Stockpiling RAP

B.5.1 The stockpiling of RAP is a very important part of the recycling process. The full benefits of comprehensive testing of the in situ asphalt layers can easily be lost if equally meticulous control of the stockpiling process is not put in place. Depending upon the variability found during testing, it may be necessary to build separate stockpiles of materials taken from different sections of the road.

B.5.2 The tendency for RAP to agglomerate will be affected by both the hardness of the bitumen in the RAP and the ambient temperature. The most effective method of stockpiling must be established by trial and error. A 250-300mm crust may form at the surface of the stockpile and this should be scalped off and reprocessed prior to recycling. Higher stockpiles should, therefore, provide more usable RAP.

B.5.3 RAP can hold up to 7-8% moisture which seriously reduces the amount of material that can be hot mixed, raises fuel costs and limits productivity. Although covering a stockpile with a waterproof sheet does keep off rain water, condensation may occur within the stockpile. Ideally RAP for hot mixing should be stored under a roof in an opensided building.

B.5.4 Stockpiled RAP destined to be used as unbound granular material may be watered to prevent agglomeration of particles in warm weather and this also aids compaction on site.

B.6 Use of RAP as unbound granular material

B.6.1 Age-hardened asphalt can be recycled as an unbound granular material. It may be produced as millings, crushed asphalt from lumps or as granulated asphalt. The RAP can be mixed with fresh aggregate to produce a particle size distribution appropriate to the layer in which it will be used. The harder the bitumen in the RAP the easier it will be to crush, handle and recompact in the new layer.

B.6.2 In contrast, an asphalt which has failed by plastic deformation will have suffered little or no bitumen hardening in the wheel paths. In the oil lane the bitumen may have softened over time whilst material outside of the wheelpaths or oil lane may have significantly age-hardened. This type of material is difficult to process and the best results can be expected by selective milling and stockpiling before reblending and adding fresh

B.7 Cold mix recycling

B.7.1 Cold mix recycling can be done at partial or full depth in an asphalt pavement with mixing carried out in-place or off-site at a central plant. The process preserves aggregate and bitumen, air quality problems are minimised and energy requirements are low. The existing pavement layers are reprocessed with the addition of fresh aggregate if this is required. During the reprocessing operation, hydraulic stabiliser, such as Portland cement or emulsified or foamed bitumen, is mixed in to produce a new material with the required properties.

B.8 Plant hot mix recycling

B.8.1 General

Hot-mix recycling is most likely to be done off-site at a central plant. Asphalt containing tar should not be recycled because of the high risk of generating carcinogenic material.

B.8.2 RAP feed to plant

To avoid blockages that will substantially reduce output, RAP should be metered into the plant through cold feed bins having the following characteristics:

a) The sides should be steeper than those of an aggregate feed bin.

b) The bottom of the bin may be longer and wider than that of an aggregate feed bin.

c) The bottom of the bin may slope downwards, to match an angled feed belt, and the end wall is sometimes left open.

d) Vibrators should not be used.

e) RAP should be delivered slowly into the cold feed bin from the front-end loader.

The level in the bin should be kept fairly low. This means that the bin must be fed more frequently than is necessary for a normal aggregate cold feed bin.

Material should not be left in the cold feed bin for more than one hour. It is more economical to run out the contents of the bin than to clear it sometime later.

B.8.3 Batch plant recycling

Because cold aggregate travels towards the heating flame in this type of plant the introduction of RAP would result in excessive smoke and other problems. The technique of conductive heat transfer, which involves the super heating of fresh aggregate and adding cold RAP via the elevator or directly into the weigh hopper minimises the likelihood of air pollution. The percentage of RAP that can be used depends upon the following factors:

a) The temperature to which the virgin aggregate is heated.

b) The temperature and moisture content of the RAP.

c) The required temperature of the final mix. Under ideal conditions, batch plant recycling can blend up to 40 per cent RAP with superheated fresh aggregate but 15 to 25% is more typical.

B.8.4 Batch mixers with a separate heating drum (parallel drum)

B.8.4.1 In this system RAP is heated in a separate drum to about 130ºC. Fresh aggregate is separately heated to a high temperature and both materials are weighed to produce the required blend in the mixing unit. The final temperature of the blend is about 160º C. Preheating allows 50% of RAP to be used in the blend, or even more if a consistent quality of output can be guaranteed.

B.8.4.1 Preheating the RAP allows the production of a more uniform mix and better control of mix temperature and this is the preferred method of recycling. However, development continues and other types of plant specifically designed for recycling bituminous materials are becoming available.

B.9 Blending with a soft bitumen

B.9.1 General

If a softer bitumen is added with the intention of bringing the blended bitumen within specification, the penetration (P) of the fresh bitumen can be calculated using equation given below:

$$LogP = \frac{A \log Pa + B \log Pb}{100}$$

where,

P is specified penetration of final blend.

Pa is penetration of RAP bitumen.

Pb is penetration of virgin bitumen.

A is percentage of RAP bitumen in the final blend.

B is percentage of virgin bitumen in the final blend. In this relationship the 'blend' is the total quantity of bitumen only, i.e. A+B = 100.

B.9.2 Limitations of bitumen blending

B.9.2.1 Bitumen in RAP recovered from a cracked asphalt will typically have a penetration of less than 15 and satisfactory blending of the new and old bitumen cannot be expected. For example, to obtain a final penetration of 80 in a blend of 60 per cent of fresh bitumen and 40 per cent RAP bitumen in which the bitumen had hardened to a penetration of 15, would require the use of a fresh bitumen with a penetration of approximately 200. It is highly likely that some fresh aggregate would only be coated with the soft fresh bitumen and this could play a dominant role in mix performance with a risk of failure through plastic deformation.

B.9.2.2 The most reliable method of obtaining a robust design with brittle asphalt is, therefore, to regard the bitumen in hardened RAP as being part of the aggregate structure and to use a 60/70 or 80/100 penetration grade bitumen, rather than a soft binder. This will prevent the possibility of plastic deformation in the new mix.

B.9.2.3 In the case of RAP from areas of plastic deformation the effect of the softer existing bitumen can be taken into account during the mix design process. Testing of laboratory and plant mix asphalt to ensure that requirements for volumetric design and Marshall properties are met, will be required just as for new material and therefore the Marshall procedures should be followed. Additional information from a performance test such as the wheel tracking test will also be very helpful in this evaluation.

B.9.2.4 The percentage of RAP that can be used will be controlled by the mixing temperature that can be achieved in the blended material. The temperature must be high enough to ensure that the fresh bitumen is at a suitable viscosity for mixing.

Annex C
(normative)

Mechanical performance evaluation of asphalt mix

C.1 Axial Loading Slab Test

The axial loading slab (ALS) test with the associated probabilistic analysis is recommended for high volume roads or applications where a level 3 rating is obtained for the rutting as a design objective.

The test involves a small slab of asphalt compacted to the specified field density by means of a slab compactor or extracted from the field. The slab is placed in a steelmould, and on a synthetic support with a known stiffness. Three types of synthetic materials (representing low,medium and high stiffness supports) can be selected to approximate the support conditions for the planned asphalt layer. The asphalt slab is repetitively loaded in the axial direction using a loading platen with a 100 mm diameter. The test is performed at three temperature levels.

The ALS test data can be used to develop a regression equation which estimates permanent deformation as a function of number of load applications, temperature and stress level. This equation can be used to evaluate the mix rut potential at different loads and temperatures. Such an analysis can be performed using a computer spreadsheet. Ideally, the ALS data and regression equation should be used in a Monte Carlo simulation to estimate the development of permanent deformation under cumulative traffic. Computer programs such as the PRORAS (Probabilistic Rut Analysis System) computer program can be used to perform the Monte Carlo analysis. The ALS test and associated PRORAS analysis system were developed to simulate field conditions as closely as possible as far as rate of loading, stress state, temperature and traffic load distribution is concerned. The probabilistic analysis takes into account the variation in loading due to random variations in temperature and load as well as traffic wander. In addition, both the daily and monthly temperature variation is taken into account in the analysis. The PRORAS system therefore provides a quantified estimate of actual rutting performance. However, because the laboratory test is accelerated and scaled down, the results still need to be interpreted in a somewhat relative manner, and comparison with similar mixes tested in the past is recommended.

C.2 Wheel Tracking Tests

Although wheel tracking tests appear to be well correlated with rutting in the field, there are at present no quantified relationships to link wheel tracking test results to rutting in the field under variable traffic loading and environmental conditions. For this reason,wheel tracking tests cannot be used to provide a quantitative estimate of rutting in the field. The test does, however, provide a reliable estimate of the rutting potential of a mix relative to similar mixes that have been tested in the past. Wheel tracking tests are particularly recommended for the evaluation of rutting performance on stone skeleton mixes, or mixes which involve modified binders, as experience has shown that these mix types cannot be properly evaluated by means of conventional test methods such as the dynamic creep and ITS. Currently, two types of wheel tracking devices are used for rutting evaluation.

C.2.1 Model Mobile Load Simulator (Mk.3 MMLS)

The MMLS is operated and distributed by the University of Stellenbosh. The MMLS differs from most wheel tracking devices in that trafficking is achieved with 4 bogies instead of a single wheel Figure C.1 MMLS Mk.3

(see Figure 6.1). Each bogie consists of a single 300 mm diameter wheel, with a maximum inflation pressure of 800 kPa and a maximum load of 2,7 kN.

A major advantage of the MMLS, as compared to other wheel tracking devices, is its high rate of trafficking: more than 10,000 simulated axle loads per hour can be applied. Another significant dvantage of the MMLS is that it can be transported to field sites for the testing of full scale asphalt pavements. The weight of the Mk.3MMLSis approximately 800 kg (including deadweight needed for 2.4 kN load per

wheel

Figure C.1 — MMLS Mk.3

C.2.2 Transportek Wheel Tracking Device

The Transportek wheel tracking device was developed to assess the rutting susceptibility of asphalt mixes and also to enable the measurement of strains under moving wheel loads. The device is used both to compact and test asphalt slabs. A segment of a steel wheel roller is used for compaction, while a solid rubber wheel (400 mm diameter, 100 mm wide) is used for rutting evaluation. Slab dimensions can be 280 by 320 mm or 350 by 660 mm, with the latter slab size being used most often for rutting evaluation. Figure C.2 shows the compaction of a slab before testing. Figure C.3 shows the wheel tracking device in operation, and Figure 6.4 shows a transverse section of a slab after wheel track testing.

Figure C.2 — Compaction of Slab for Wheel Tracking Test

Figure C.3 — Wheel Track Testing

Figure C.4 — Deformation of Slab after a Wheel Tracking Test

Typically, the level of compaction aimed for in the wheel tracking test will be the same as that which is specified for field compaction. The level of compaction achieved during the compaction process can be accurately controlled with the Transportek wheel tracking device. However, the actual density achieved is dependent on the accuracy and applicability of the MTRD and mass calculations used to determine the amount of material that should be used to compact a slab. Itis therefore recommended that more than one slab be compacted for the monitoring of the voids and density that is achieved after slab compaction. If the wheel tracking tests are carried out at the design stage, it is imperative that the MTRD of the design mix and that of the eventual plant mix be compared to ensure that the rut performance of the constructed mix is comparable to that obtained during wheel tracking tests. Ideally, the plant mix should also be tested in the wheel tracking device to ensure that the properties of the design mix are the same as those of the plant mix.

Because wheel tracking test results are evaluated in a relative manner, data post-processing is minimal and therefore the cost of testing is significantly lower than that of the axial loading slab test. The test is therefore recommended for applications where rutting has a medium to high importance as a design objective, but which do not justify the higher cost associated with a quantitative estimate of rutting performance.

The standard test protocol for the Transportek wheel tracking device is to performthe test at 60°C and at a load of 600 kg (which equates to a contact pressure of approximately 900 kPa). For this test protocol, the limits shown in Table C.1 can be used as a tentative guideline to the evaluation of rutting performance.

Table C.1 — Interim guidelines for the interpretation of wheel tracking results

Repetitions to 10 mm Rut Depth	Mix Classification
< 2500	Poor
2500 - 5000	Medium
> 5000	Good

C.4 Recommended Test Procedure for Rutting Evaluation

The three approaches to rutting evaluation provides designers with a flexible and cost effective approach to rutting evaluation which can be adapted to suit most design situations. Designers should become familiar with the advantages and disadvantages of the different rut evaluation procedures so that the most cost effective and appropriate evaluation method can be selected for a particular design situation. Table C.2 provides a recommended test selection matrix for different design situations.

Table C.2 — Recommended rut evaluation tests

Mix Type	Design Objective Rating for Rutting		
	1	2	3
Sand Skeleton Mixes Dense Graded Mixes LAMBS	Expert System Evaluation	Wheel Tracking Test	Wheel tracking test or Axial Loading Test with PRORAS analysis and rut prediction
Stone Skeleton Mixes Stone Mastic Asphalt Open Graded Mixes	Spatial composition	Spatial composition	Wheel tracking test or Axial Loading Test with PRORAS analysis and rut prediction

C.5 Performance testing for fatigue evaluation of asphalt wearing courses

Apart fromthe four point bending beam fatigue test, very fewlaboratory tests provide a consistent evaluation of the fatigue performance of wearing courses. As four point bending beam fatigue tests are relatively expensive to perform, they are therefore mostly used for applications on high trafficked roads. For the design of wearing courses on pavements with low traffic volumes, where expensive performance tests are not always warranted, the indirect tensile strength (ITS) test parameters, coupled with the evaluation of binder durability, can be used to determine whether there is a risk of premature fatigue failure and whether four point bending beam fatigue tests are warranted. Table C.3 provides guidelines for the interpretation of ITS data (measured at 25°C) for the fatigue evaluation of asphalt wearing courses.

Table C.3 — Guidelines for the Interpretation of ITSresults for fatigue performance evaluation

Relative Fatigue Performance	ITS (kPa)	ITS strain at Maximum Stress (%)
Good	< 1000	> 2.2
Medium	1000 to 1400	1.5 to 2.2
Poor	>1400	< 1.5

Note: The above recommendation is only valid for relatively thin wearing courses. For asphalt base courses, the ranking would be the opposite of that shown above, where the designer should strive to achieve high ITS values.

Wearing coursemixes for which Table C.3 suggests a poor fatigue performance should be further evaluated bymeans of a detailed binder evaluation and an evaluation of the spatial composition. In such instances, designersmayalso consider performing bending beam fatigue tests to validate the indication given by the less exact ITS test.

C.5 Four Point Bending Beam Test

Figure C.5 shows a schematic representation of the four point bending beam fatigue test. During the test, rectangular beam specimens are subjected to a repeated load. Tests are normally performed at 5°C, using a sinusoidal load with a frequency of 10 Hz. Two modes of loading can be applied. In the constant strain mode of loading, the strain at the bottom of the beam is preselected and kept constant for the duration of the test. Failure is defined as the point at which the stiffness of the beam is reduced to 50 per cent of its initial stiffness. In the constant stress mode of loading, the load is kept constant and the strain is allowed to vary. The test is terminated when the beam fails (i.e. cracks). The constant strain test is normally used for thin (< 70 mm) surfacing applications, the constant stress mode being used for the evaluation of thick (>70mm) asphalt bases.

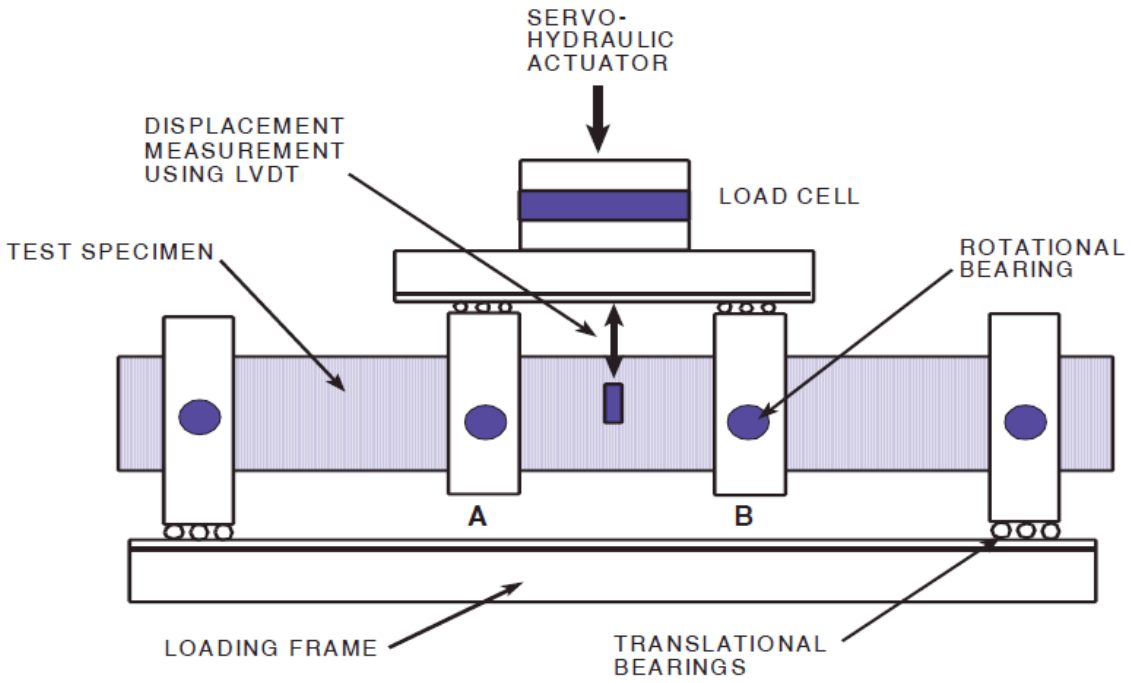

Figure 7.1 Schematic Illustration of the Four Point Bending Beam Test

Figure C.5 — Compacted Sample using Rolling Wheel Compaction

Figure C.6 — Compacted Sample using Rolling Wheel Compaction

Owing to the small strains induced during testing, as well as the dynamic nature of the loading, bending beam tests require sophisticated loading and data acquisition equipment. For this reason, bending beam fatigue tests are normally performed in specialist laboratories.

Bending beam samples are cut from a larger slab of material which is normally prepared using a rolling wheel compactor (see Figure 7.2) with the compaction effort and quantity of material being controlled so as to allow the design voids content to be attained. Approximate dimensions of beam samples are: 400 mm long x 60 mm wide x 50 mm high. Samples are preconditioned at the testing temperature by placing them in an oven set to the test temperature, several hours before they are tested.

C.5.1 Interpretation of Results

Table C.4 provides guidelines for the interpretation of four-point bending beam fatigue results. It should be noted that the limits shown in Table C.4 pertain to constant strain tests performed at 5°C and at a loading rate of 10 Hz. In these tests, the point of failure was defined as the number of load repetitions at which the original beam stiffness had decreased by 50 per cent. The repetitions to failure referred to in Table C.4 cannot therefore be interpreted as the number of axles before the onset of fatigue cracking.

Table C.4 — Guidelines for the interpretation of Bending Beam fatigue data (Constant stain)

Relative Fatigue Performance	Number of Repetitions to Failure for Strain Regime (Millions) @ 5 C		
	Low Strain (180 to 230)	Medium Strain (320 to 370)	High Strain (380 to 430)
Good	> 2.4	> 0.13	> 0.06
Medium	1.0 to 2.4	0.03 to 0.13	0.02 to 0.06
Poor	< 1.0	< 0.03	< 0.02

In an HMA design, Table C.4 can be used as follows:

Step 1: The designer should calculate the expected strain level for the given design situation. Conventional multilayer elastic models can be used for this calculation. The calculated strain level is used to determine the approximate working strain regime for the design situation, as defined in Table C.4.

Step 2: Perform four-point bending beam tests at a strain level which falls inside the strain range shown in Table C.4 for the appropriate strain regime. For, example, for the low strain regime, tests can be performed at 220 microstrain. It is recommended that a minimum of 3 beams be tested at the appropriate strain regime.

Step 3: Use the bending beam test results together with the ranges shown in Table C.4 to evaluate the relative fatigue performance of the design mix. Use Table C.5 to estimate the design fatigue life of the design mix.

As can be seen from Table C.5, the fatigue life increases dramatically if the working strain limit is below approximately 230 microstrain. For designs for high traffic volumes, it is therefore strongly advised that the recommended support conditions be observed to ensure that the mix will operate in the low strain regime.

Mixes that operate in the low strain regime are likely to fail because of climatic influences (embrittlement due to oxidation etc.) rather than traffic loading. The fatigue life is thus likely to be coupled to time rather than to the number of load applications. Experience indicates that a typical effective fatigue life of thin asphalt mixes is between 8 and 12 years. Mixes with a good bending beam fatigue performance can be expected to last to the upper limit of this range, while those with a poor fatigue performance may last only to the lower limit.

Designers should be aware that mixes that operate in the medium to high strain regimes are very sensitive to variations in load and support conditions. It is recommended that mixes which are designed for these conditions, and which require a low risk of failure, be improved either by modification with bitumen rubber, or by increasing the binder content to ensure greater flexibility and durability.

Table C.5 — Guidelines for fatigue life estimation (Constant stain)

Quality Characteristic	**Traffic Category**		
	heavy	medium	light
Air voids content (%)	4.0	4.0	4.0
Voids in mineral aggregate (% min.)			
4.75-mm grading	17.0	17.0	17.0
9.5-mm grading	15.0	15.0	15.0

12.5-mm grading	14.0	14.0	14.0
19-mm grading	13.0	13.0	13.0
Stabilometer value (min.)			
12.5mm and 19 mm gradings	37	35	32
Swell	less than 0.762 mm (0.030 in.)		

Notes:

1. Although not a routine part of this design method, an effort is made to provide a minimum percent of air voids of approximately 4 percent.

2. All criteria, and not stability value alone, must be considered in designing an asphalt paving mix.

3. Hot mix asphalt bases that do not meet these criteria when tested at 60°C (140°F) are satisfactory if they meet the criteria when tested at 38°C (100°F) and are placed at 100 mm (4 in.) or more below the surface. This recommendation applies only to regions having a range of climatic conditions similar to those prevailing throughout most of the United States. A different lower test temperature may be considered in regions having more extreme climatic conditions.

4. Traffic classifications:

Light: Traffic conditions resulting in a Design EAL $<10^4$

Medium: Traffic conditions resulting in a Design EAL between 10^4 and 10^6

Heavy: Traffic conditions resulting in a Design EAL $>10^6$

Table 7.3 Guidelines for Fatigue Life Estimation (constant strain)

Relative Fatigue Performance*	Approximate expected fatigue life before the onset of fatigue cracking		
	Low Strain (180 to 230)	Medium Strain (320 to 370)	High Strain (380 to 430)
Good	> 15 MESA** or 12 years	2 MESA	0.5 MESA
Medium	8 to 15 MESA or 10 years	0.3 to 1.0 MESA	0.06 to 0.5 MESA
Poor	< 8 MESA or < 8 years	< 0.3 MESA	<0.06 MESA

* As determined by bending beam fatigue tests and through Table 7.2.

** MESA = Millions of Equivalent Standard Axles.

As determined by bending beam fatigue tests and through Table 7.2.

** MESA = Millions of Equivalent Standard Axles.

C.6 Fatigue Evaluation of HMA Bases

Fatigue testing for HMA bases is normally performed in the constant stress mode of testing. To date, no constant stress tests using modern testing equipment have been performed in South Africa. Consequently guidelines for the interpretation of constant stress test results cannot be provided at this stage. Until more experience is gained in the interpretation of constant stress data, it is recommended that HMA base mixes be tested in the constant strain mode, at a strain level of 220 microstrain. The fatigue performance of the mix can then be evaluated in a relative manner using Table C.4. Although this approach does not provide a reliable estimate of mix fatigue life, it may alert designers if the mix is likely to have unusually low fatigue properties.C.7 Indirect Tensile Strength (ITS) Test

C.7.1 Test Description

The indirect tensile strength (ITS) test is commonly used to evaluate the cohesive strength of asphalt mixes. This property can be used to evaluate tensile strength (related to toughness and durability) and is also an important component of rutting resistance in the medium temperature range. The test does not require sophisticated testing equipment and can be performed on briquettes manufactured in the laboratory, as well as on cores obtained from the field.

In the ITS test the sample is loaded on its diametral axis, as illustrated in Figure C.7 shows a sample positioned in the testing frame. The widths of the loading strips are prescribed by ASTM D41236 and are 13 mm for a 102 mm diameter specimen and 19 mm for a 152 mm diameter specimen. During testing, the sample is loaded at a fixed rate of loading (a rate of 50 mm per minute is typically used) until a significant loss in applied load is noted. The peak load is used to calculated the indirect tensile strength. The formula for calculation of the ITS (in kPa) is as follows:

$$ITS = \frac{2.0\ P_{ult}}{\pi\ .\ t\ .\ D}$$

Where:
P_{ult} = Ultimate applied load, in kN;
t = Thickness of the specimen, in mm, and
D = Diameter of the specimen, in mm.

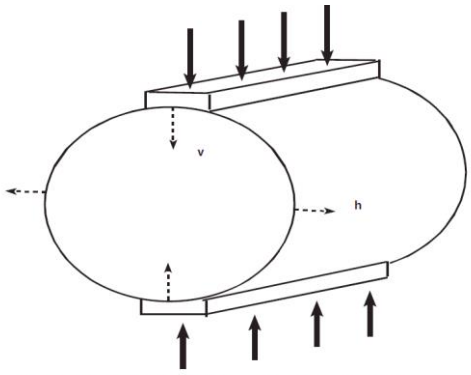

Figure 8.1 Schematic illustration of the Indirect Tensile Test

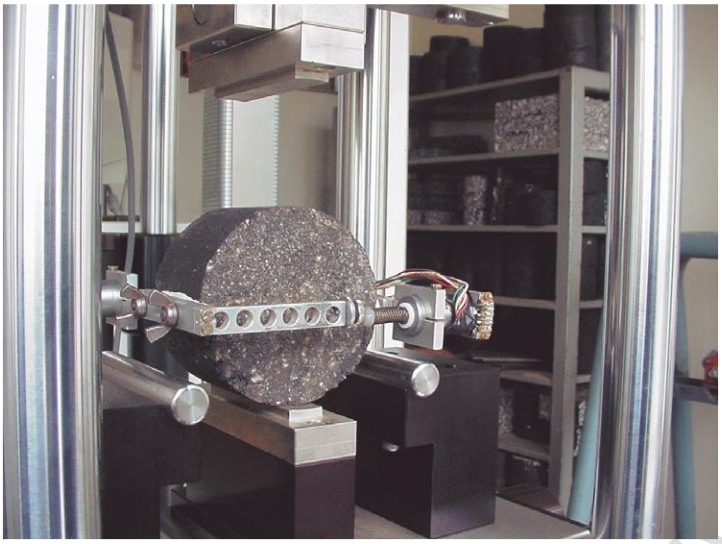

Figure C.7 — Sample positioned for ITS testing

Figure 8.2

ASTM D 41236 prescribes sample dimensions and preparation for the indirect tensile strength and resilient modulus test. Specimens for the ITS test can be prepared using laboratory compaction techniques such as the Marshall or Gyratory compaction devices. Specimens can also be obtained from the field bymeans of coring. Specimens should have a thickness of at least 51mm and a diameter of 102 mm or larger for aggregates of up to 25 mm maximum size. For aggregates with a maximum size of between 25 and 38 mm, the minimum thickness and diameter should be increased to 76 mm and 152 mm, respectively. Care should be taken to ensure that cores have smooth parallel surfaces.

C.7.2 Interpretation of Results

The indirect tensile strength of asphalt mixes provides an indication of the cohesive strength of asphalt mixes and is therefore strongly influenced by the characteristics of the binder. As an indicator ofmix cohesion, ITS values provide an overall indication of mix stability in the low to midtemperature range (10 to 40°C) and can be expected to be related to rutting resistance as well as durability and stripping potential.

The minimum value for ITS in South Africa is 800 kPa. However, designers should be aware that limited field studies have suggested that rutting potential tends to increase for ITS values below approximately 1000 kPa. At the same time, ITS values in excess of 1700 kPa may indicate a tendency to brittleness and low flexibility. An ideal range for ITS values would seem to be between 1100 and 1500 kPa. Table C.6 shows some statistical parameters that are typically associated with ITS values. The comments in Table C.6 refer to relatively thin asphalt wearing courses.

Table C.6 — Typical ITS Results

Statistical Parameter	**Typical ITS results (in kPa) at 25°C**	**Comments/Interpretation****
95th percentile value	1650	Value s above this may be indicative of brittleness or poor flexibility of wearing courses
75th Percentile Value	1200	Values close to this are indicative of good rutting performance

Average	1100	None
25th Percentile Value	900	Values below this may be indicative of poor rutting or stripping performance
* Based on 33 observations. ** SMA mixes and mixes manufactured with some polymer-modified or bitumen-rubber binders may have low ITS values and still exhibit good performance. The comments and interpretations may therefore not be applicable for such mixes.		

C.8 Constant Head Permeability Test

C.8.1 Compaction of Briquettes

Combine the aggregates in the correct proportions to meet the final design grading. Add the required amount of binder to the aggregate after heating the ingredients to the correct temperature as defined in Method C2 of TMH1-1986. Mix thoroughly. The mixture is compacted by applying 70 blows with the Marshall hammer, 35 blows to each side of the specimen. Remove the specimen from the mould by means of an extraction jack after cooling sufficiently. Measure the density of the briquette as described in Method C3 and C4 of TMH1-1986.

C.8.2 Preparation for permeability test

Apply a thin layer of silicone sealant around the perimeter of the compacted briquette. Leave to dry, then place in a steel casting mould with 122-125 mm internal diameter and 93 mm high which results in a 12.5 mm wide void around the sample (Figure C.8). Seal the void between the sample and the inside of the mould using Plaster of Paris taking care not to contaminate the upper and lower surfaces of the briquette with the Plaster of Paris. Allow the Plaster of Paris to dry and apply silicone sealant to the upper and lower exposed edges of Plaster of Paris between the mould and the sample. After the silicone sealant has cured, steel cover plates fitted with an “O” ring to form a watertight seal on the top and bottom edges of the mould, and also fitted with two ball valves, one for water entry and one to act as a bleed valve, are bolted on to the mould. The bottom ball valve of the mould is connected to a constant head permeameter with a water head of 1.0 m and readings of the volume of water coming out of the mould are taken at 5-minute intervals. The test is stopped when the volumes measured for each 5-minute interval are reasonably constant.

Figure C.8 — Typical Set-Up for the Laboratory Water Permeability Test

The permeability (expressed in per square metre per hour) of the asphalt sample is determined by using the following formula:

Permeability = Q/A where Q = flow in /h and A = area in m2

C.9 Moisture Sensitivity (Modified Lottman Test)

C.9.1 Test Description

The Modified Lottman test for measurement of moisture sensitivity relies on indirect tensile strengths measurements taken before and after conditioning by freeze-thaw cycles. The test is performed according to the ASTM D48676 protocol (note: an alternative method is provided in AASHTO 2838, but ASTM D4867 is preferred). In the test, six samples are compacted to within a void content range of 6 to 8 per cent (or to the field voids) and partially saturated with water (saturation limit of between 55 and 80 per cent). Three of the six samples are frozen for at least 15 hours and subsequently immersed for 24 hours in a hot bath set at 60°C (i.e. “conditioned” samples). All six samples are then brought to a constant temperature and their indirect tensile strengths determined. The ratio of the indirect tensile strengths of the conditioned and unconditioned samples is referred to as the tensile strength ratio (TSR).

C.9.2 Interpretation of Results

For routine mix design purposes, a minimum TSR of 0.7 is usually specified. For mixes in high rainfall areas and high traffic applications, a minimum TSR of 0.8 is recommended. Table C.7 provides TSR criteria based on the permeability of the mix and the climate in which the mix will operate.

Table C.7 — TSR criteria based on mix permeability and climate

Climate	Permeability		
	Low	Medium	High
Dry	0,60	0,65	0,70
Medium	0,65	0,70	0,75
Wet	0,70	0.75	0,80

C.10 Compressive strength of hot mi asphalt

The compressive strength of hot mix asphalt can be determined by the method given in AASHTO, T 167-10

Bibliography

[1] MS-2, Asphalt Mix Design Methods

[2] Overseas Road Note 19, A guide to the design of hot mix asphalt in tropical and sub-tropical countries

[2] Interim guidelines for the design of hot-mix asphalt in south Africa

