OVERSEAS ROAD NOTE 19

A guide to the design of hot mix asphalt in tropical and sub-tropical countries

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OVERSEAS ROAD NOTES

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Foreword

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-mixed asphalt (HMA) i.e. a mixture of aggregate materials bound together with bitumen. The development of techniques for designing and constructing such surfaces has relied primarily on empirical methods rather than on a fundamental understanding of the physical interactions that take place. Such surfaces have proved to be reasonably successful, especially in temperate climates where the climatic conditions are not severe and where sufficient empirical evidence has been collected for reliable and reproducible designs to have evolved.

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.

In tropical and sub-tropical countries the performance of HMA has often been disappointing, with road surfaces sometimes failing within a few months of construction and rarely lasting as long as hoped. Under the high temperature conditions experienced in these countries, bitumen, which is a visco-elastic material, can become very soft. It can also undergo relatively rapid chemical changes that cause many of the desirable properties of the HMA to be degraded or lost altogether. Thus developing a design method for HMA surfacing material that ensures good long-term performance under a wide range of tropical conditions has provided a challenge for research engineers and scientists.

This Road Note has been based on the experience of TRL Limited and collaborating organisations throughout the world. Most of this experience has been gained in carrying out a comprehensive, co-ordinated and long-term series of research projects as part of the 'Knowledge and Research' programme of the United Kingdom's Department for International Development. The research showed that the behaviour of asphalt surfaces in tropical and sub-tropical environments was frequently contrary to expectations and has given rise to a paradigm shift in our understanding of road behaviour. The outcome of the research has provided a new understanding of the problems associated with the use of HMA in hot tropical climates and has resulted in the development of some practicable methods for overcoming these problems.

This Road Note is aimed at engineers responsible for roads and gives guidance on the design, manufacture and construction of HMA pavement materials in tropical and sub-tropical climates. The HMA requirements are described for different traffic loading categories, including severely loaded sites such as climbing lanes. The procedures take into account the fact that many countries have limited facilities for designing bituminous mixes and therefore need to use commonly available or inexpensive equipment. The Road Note complements and, in many parts, also updates Overseas Road Note 31 (TRL, 1993 which gives recommendations for the design and construction of new road pavements but which also includes chapters on the design of pavement layers).

The methods of design that are described in this Road Note remain firmly based on practical experience. Because of the wide diversity of road building materials, climates, vehicle flows and vehicle loading characteristics that may be encountered, the Road Note also makes reference, where necessary, to the standards and guidance documents produced by other international authorities. The importance of local knowledge and the judgement of experienced engineers should, however, never be overlooked.

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Department for International Development

London

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1 Introduction

1.1 This Road Note gives guidance on the design, manufacture and construction of hot premixed bituminous pavement materials, or hot mix asphalt (referred to as HMA in the remainder of this Road Note) in tropical and sub-tropical climates. Mix requirements are described for different traffic loading categories including severely loaded sites such as climbing lanes.

1.2 The Road Note is aimed at highway engineers responsible for the design and construction of plant-mixed bituminous surfacings. It complements Overseas Road Note 31 (TRL, 1993), which gives recommendations for the design and construction of new road pavements, and presents revised recommendations for the HMA designs given in that publication.

1.3 An important aspect of this Guide is that it addresses the actual modes of failure that occur in HMA surfacings in tropical and sub-tropical environments and which are frequently contrary to those assumed to be the most prevalent.

1.4 The recommendations have been based on the results of research into the performance of full-scale surfacing trials carried out with the co-operation of Ministries of Works in several tropical countries. Reference has also been made to standards and design guides produced by other international authorities with responsibilities for road construction in hot climates.

1.5 The wide diversity of road building materials, climates, vehicles traffic flow and vehicle loading characteristics means that, of necessity, this Road Note should be treated as a guide and the importance of local knowledge should not be overlooked.

1.6 Figure 1.1 shows the layers which may be present in a road pavement and which may be bound with bitumen. The descriptions given of the individual layers are those used throughout this Road Note. Where thick HMA surfacing layers are required, they are normally constructed with a wearing course laid on a binder course.

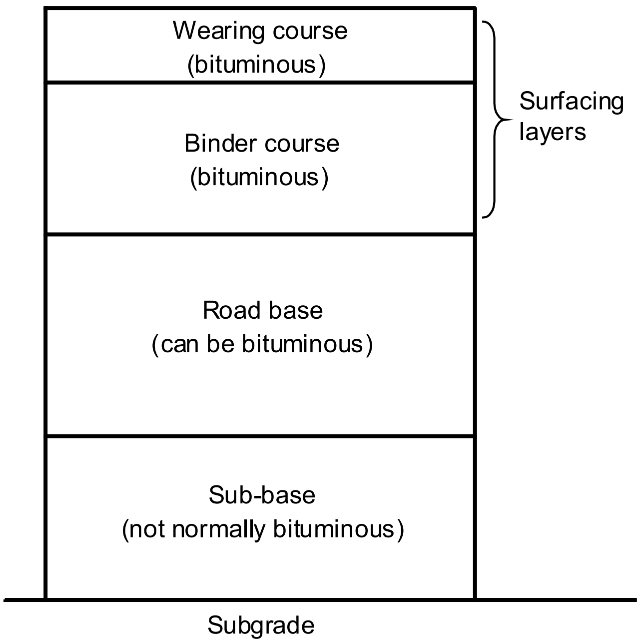


Figure 1.1 Pavement layers which may be bitumen bound

1.7 HMA wearing courses are the most critical layer in a pavement structure and must be of high quality and have predictable performance. Typically HMA wearing courses need to possess the following characteristics:

i high resistance to deformation;

ii high resistance to fatigue and the ability to withstand high strains i.e. they need to be flexible;

iii sufficient stiffness to reduce stresses in the underlying layers to acceptable levels;

iv high resistance to environmental degradation i.e. good durability;

v low permeability to prevent the ingress of water;

vi good workability to allow adequate compaction to be obtained during construction;

vii sufficient surface texture to provide good skid resistance in wet weather; and

viii predictable performance.

1.8 Designing a mix having all of these characteristics will often result in conflicting design indicators. For example, high ambient temperatures reduce the stiffness of dense and durable mixes making them more prone to plastic deformation, but more open-graded mixes designed to resist plastic deformation will be vulnerable to accelerated bitumen oxidisation and hardening and, hence, be less durable.

1.9 Different mix designs are also sometimes necessary for different lengths of the same road. For example, mixes suitable for areas carrying heavy, slow-moving traffic, such as a climbing lane, will be unsuitable for flat, open terrain where the traffic moves more rapidly. A mix suitable for the latter is likely to deform on a climbing lane and a mix suitable for a climbing lane is likely to possess poor durability in flat terrain.

2 Composition of HMA

Components of a mix

2.1 The types of HMA most frequently used in tropical countries are manufactured in an asphalt plant by hot-mixing appropriate proportions of the following materials:

i coarse aggregate, defined as material having particles larger than 2.36mm;

ii fine aggregate, defined as material having particles less than 2.36mm and larger than 0.075mm;

iii filler, defined as material having particle sizes less than 0.075mm, which may originate from fines in the aggregate or be added in the form of cement, lime or ground rock; and

iv a paving grade bitumen with viscosity characteristics appropriate for the type of HMA, the climate and loading conditions where it will be used.

Note: It should be noted that the Asphalt Institute and several international authorities define coarse and fine aggregate with reference to the 2.36mm sieve whilst others, including AASHTO, refer to the 4.75mm sieve. This should be taken into consideration when specifying particular aggregate tests.

Types of HMA in common use

2.2 Two generic types of HMA are presently used in countries with tropical climates. These are:

i Mixes in which traffic stresses are transmitted mainly through an aggregate structure which has a continuous particle size distribution. Asphalt Concrete and Bitumen Macadam are examples of this type.

ii Mixes in which stresses are passed through the fines/filler/bitumen matrix. In these mixes the aggregate particle size distribution is discontinuous or 'gap-graded'. Hot Rolled Asphalt is in this category.

Asphalt Concrete (AC)

2.3 This is by far the most common type of HMA used in tropical countries and it is usually designed by the Marshall Method (Asphalt Institute, 1994). The material has a continuous distribution of aggregate particle sizes which is often designed to follow closely the Fuller curve to give the maximum density after compaction. However, such a dense structure makes AC sensitive to errors in composition and the effect of this becomes more critical as traffic loads increase.

Bitumen Macadam

2.4 This type of HMA, commonly known as Dense Bitumen Macadam (DBM), is similar to AC except that the skeleton of the compacted aggregate tends to be less dense. In Britain, where it is now known as Close Graded Macadam (BS 4987, 1993), it has traditionally been made to recipe designs and has also been used with success in tropical environments. This HMA will be referred to as DBM in this Road Note.

2.5 Recipe specifications and the necessary compliance testing are simple to use and to implement, but the transfer of recipe designs between countries having different climates, materials and traffic loading characteristics cannot be recommended because there is no simple procedure for adequately assessing the effects of these differences. In addition, because recipe specifications are based on historical performance, modifications to the specifications tend to be delayed responses to some change in conditions. Most authorities prefer to have a test procedure that ensures satisfactory performance at all times and hence recipe specifications are not commonly used.

Hot Rolled Asphalt

2.6 Hot Rolled Asphalt (HRA) has been used extensively on heavily trafficked roads in Britain over many years and also in modified forms in South Africa and Indonesia. However, as the severity of traffic loading has increased there has been a significant increase in the incidence of rutting. This type of mix is no longer recommended for heavily trafficked roads in South Africa and its use is diminishing in the UK. In Australia HRA is recommended for residential streets because the mix has good workability and it is easy to achieve an impermeable layer.

2.7 In the UK the coarse aggregate content of HRA wearing courses is typically 30 per cent. Bitumen coated chippings must be spread and rolled into the surfacing during construction to provide good skid resistance. This makes the material relatively expensive. HRA can be made to a less expensive design suitable for many roads in tropical climates by increasing the coarse aggregate content to between 45 and 55 per cent. This minimises the quantity of the relatively expensive sand/filler/bitumen mortar and avoids the need to apply coated chippings.

2.8 HRA has several advantages compared to AC. It is less sensitive to proportioning, making it easier to manufacture, and it is also easier to lay and compact. It requires fewer aggregate sizes and therefore fewer stockpiles and cold feed bins.

2.9 Unlike AC mixes, the aggregate particle size distribution is discontinuous and is referred to as being ‘gap-graded’. It is the properties of the sand/filler/bitumen matrix that determines the performance characteristics of the mix. The gap in the particle size distribution is obtained by limiting the quantity of aggregate particles between 2.36mm and 0.6mm in size. This requirement is relatively easy to comply with when fine pit-sand is available but can otherwise be difficult to achieve, especially where the available rock is difficult to crush.

2.10 Authorities who are considering the use of HRA should refer to the appropriate Australian and South African specifications.

Other types of mixes

2.11 The wear and tear on wearing course layers is often severe and these layers need to be replaced periodically to maintain desirable surface characteristics such as high skid resistance. Commercial companies have developed ‘thin surfacing’ mixes which are suitable for this type of application. These proprietary materials sometimes contain a modified bitumen, and may also include a high filler content or fibres. For rehabilitation purposes, some are able to provide a limited amount of correction to a deformed surface, but this is only applicable where the existing wearing course is stable and not deforming plastically. It is also important that the existing pavement is structurally sound and that there is little or no full depth cracking in the asphalt layer.

3 Factors affecting HMA design

Modes of failure of HMA surfacings

3.1 Traditionally HMA has been designed to resist three main modes of deterioration. These modes of deterioration are:

i fatigue cracking;

ii plastic deformation; and

iii loss of surfacing aggregate.

Cracking in HMA surfacings

3.2 Fatigue cracking results from the cumulative effect of horizontal tensile strains generated by applications of heavy vehicle loads and is expected to be initiated at, or near to, the bottom of the HMA layer where the induced tensile strains are greatest. However, investigations carried out by TRL in co-operation with Ministries of Works in several countries have shown that this type of fatigue cracking is comparatively rare and is virtually always preceded by cracking which initiates at the surface of the layer (Rolt *et al*., 1986, Smith *et al*., 1990, Strauss *et al*., 1984, Dauzats and Linder, 1982). It is now generally accepted that 'top down' cracking occurs in many countries including those with more temperate climates (Nunn *et al*., 1997).

3.3 'Top-down' cracking is associated with age hardening of bitumen in the top few millimetres of the wearing course and, in the tropics, can develop relatively early in the expected life of the surfacing. The important point is that it is not necessarily a sign of structural inadequacy. The hardened 'skin' of the surfacing is very brittle and may crack as a result of thermal or traffic induced strains or by a combination of the two. The fact that widespread 'top-down' cracking often occurs in asphalt surfacing on untrafficked areas of airfields points to the importance of environmental effects and thermal stresses. Figure 3.1 shows an example of age hardening. In this dense AC wearing course, located in a seasonally hot and dry (but not extreme) climate, a steep viscosity gradient has developed in the bitumen within two years of construction. Severe hardening is apparent in the top few millimetres of the material with the viscosity of the bitumen increasing from approximately 4.85 to 6.7 log poises (or 7.5×10^4 to 5×10^6 poises). Over the range studied the increase in viscosity at the surface was found to be independent of the percentage of voids in the material itself.

3.4 One way of improving the durability of HMA is to increase the bitumen content. This reduces the air void content and the rate of oxygen absorption (Dickinson, 1984) and any surface cracking in the

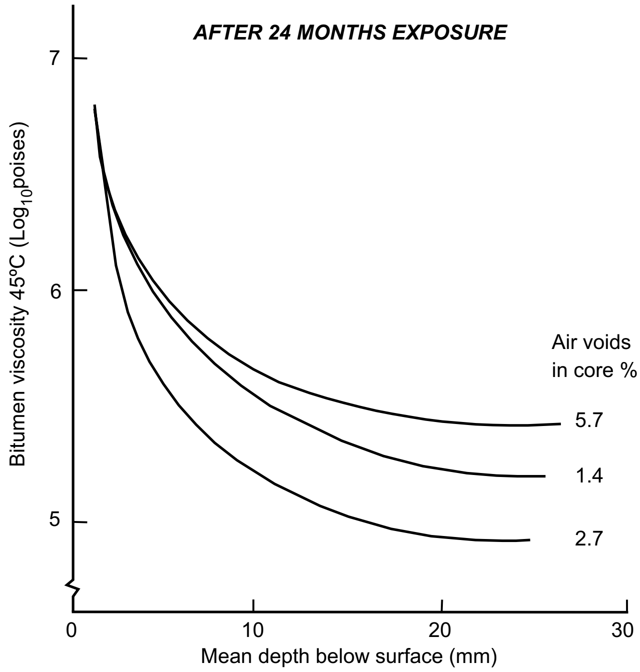


Figure 3.1 Bitumen viscosity versus depth in cores taken from a road site

bitumen-rich mix is likely to remain shallow for some considerable time. However, using a bitumen-rich mix on roads carrying more than light traffic introduces a high risk of more serious failure through plastic deformation.

Failure of asphalt surfacings by plastic deformation

3.5 Plastic deformation in HMA surfacings is the most serious form of failure because the affected material must be removed before the road can be rehabilitated. It is usually associated with an underestimate of the degree of secondary compaction that occurs under heavy traffic which reduces the air voids content, or voids in the mix (VIM), to a critical level at which plastic deformation occurs relatively rapidly.

3.6 The relationship between *in situ* VIM and asphalt deformation observed on severely loaded sites in countries experiencing high road temperatures is illustrated in Figure 3.2 It can be seen that it is necessary to ensure that VIM remains greater than 3 per cent if plastic deformation is to be avoided. This is in agreement with the recommendations of the Asphalt Institute (Asphalt Institute, MS-2, 1994).

3.7 When the VIM in an AC layer decreases to less than approximately 3 per cent, stress transfer, which was occurring through stone to stone contact in the coarse aggregate, switches to the bitumen-fines component in the mix. As secondary compaction continues, stone to stone contact is increasingly reduced until plastic deformation occurs.

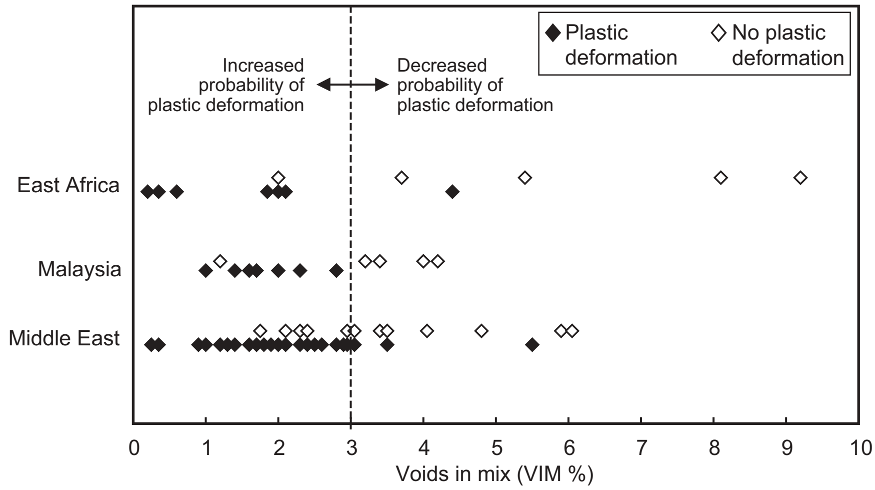


Figure 3.2 Occurrence of plastic deformation in AC wearing courses

3.8 Typical relationships between the rate of reduction in VIM and traffic on a climbing lane are shown in Figure 3.3 (Hizam and Jones, 1992).

Loss of surfacing aggregate, or fretting

3.9 This is often associated with the scrubbing action of vehicle tyres and may develop because of:

i the use of an incorrect aggregate particle size distribution, segregation, or inadequate compaction. These can result in a permeable surface leading to rapid embrittlement of the bitumen and stones breaking away from the surfacing layer;

ii the use of a low bitumen content making the mixture less durable;

iii the general embrittlement of the bitumen near the end of the design life of the wearing course;

iv 'stripping' resulting from ingress of water and poor adhesion between the bitumen and aggregate particles.

Effects of vehicle characteristics

Axle loads and vehicle speeds

3.10 Traffic loading for pavement design purposes is expressed in terms of equivalent standard axles (esa). As the design esa increases, so the thickness of the HMA layers increase in order to accommodate the greater cumulative loading. Whilst the magnitude of axle loads are important it is the characteristics and pressure of the tyre that have most influence on the performance of HMA.

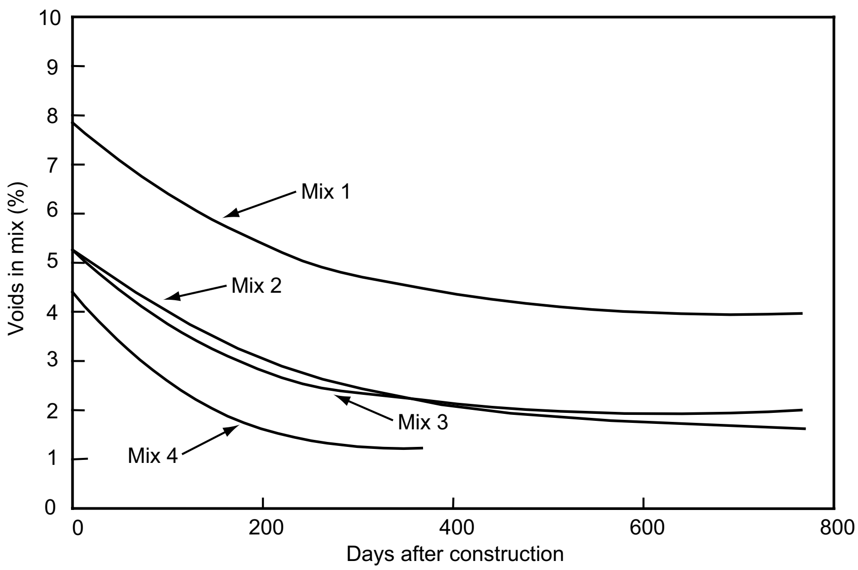


Figure 3.3 Reduction in VIM in the wheelpath of AC wearing courses designed by Marshall procedure

3.11 Vehicle speeds determine the loading time which, in turn, will also affect the performance of HMA surfacings. Under slow moving heavy vehicles the longer loading time results in an effective reduction in the stiffness of the HMA and increased secondary compaction. Therefore, an HMA which is suitable for climbing lanes will retain higher VIM when used on flat terrain, where vehicles speeds are higher, and will be less durable.

Type of tyre

3.12 Greater use of radial ply tyres has increased the severity of traffic loading. On roads which carry high traffic it is common for the vehicles to be 'channelled' and to form distinct wheelpaths. Where this concentrated loading causes even a shallow rut to form, the traffic loading can become even more concentrated. In the past, cross ply (or biased) tyres tended to 'climb out' of any rut that formed, thereby distributing vehicle loads over a relatively wide wheelpath. However, radial ply tyres tend to run in the bottom of the ruts (Gillespie *et al.*, 1993) thereby producing much narrower wheelpaths and more intensive traffic loading. The complexity of predicting the effects of traffic loading has also been increased by the introduction of wide-based single tyres (sometimes called super singles), whose damaging effect appears to depend, not surprisingly, upon their width.

Tyre pressures

3.13 Tyre pressures have also increased significantly over recent years and this has resulted in more severe loading at the road surface. The unpublished results of a survey, carried out in 1987 by the Ministry of Works and Housing in Kenya showed that, typically, the tyre pressure of 0.48 MPa (70 psi) used at the AASHO Road Test (HRB, 1962) approximated to the lower 10 percentile tyre pressure of heavy commercial vehicles in Kenya. The mean value recorded during the study was 0.7MPa (102 psi) and the highest was approximately 1.03MPa (150 psi).

Maintenance

3.14 Roads with HMA surfacings are normally designed for 10 to 20 years of trafficking, with 15 years being a typical target. Frequently roads with HMA surfacings in tropical climates suffer cracking long before their design lives have been reached, but the running surface can remain relatively smooth until the cracks propagate to the bottom of the HMA. At this stage, water ingress into lower granular materials usually leads to rapid structural damage. The problem then is that maintenance funds cannot be raised quickly enough to prevent the need for major rehabilitation work.

3.15 It is therefore important that maintenance is carried out before significant damage occurs to the roadbase. The optimum timing and frequency of maintenance is difficult to predict sufficiently accurately from a project level analysis since it will depend upon many factors such as the properties of the HMA surfacing material, climate and traffic but, typically, a seal is often required within five years of laying an HMA surfacing.

3.16 In countries where it is unlikely that funds will be available to carry out such maintenance, it will be cost effective to seal the HMA as part of the construction process. If this procedure is adopted then the use of binder course HMA will be acceptable and may be cheaper than a finer wearing course material. A binder course mix normally has a higher percentage of large sized aggregate than a wearing course mix and will be more resistant to the embedment of chippings in early life. However, whichever type of mix is used, the design of a surface dressing must take into account the hardness of the new HMA surfacing (TRL, 2000). A short delay may be necessary to allow the surfacing to harden before the dressing is applied.

3.17 A surface dressing placed soon after construction will prevent the formation of a steep bitumen viscosity gradient in the surface of the wearing course and significantly reduce the risk of early cracking. However, if such a seal is constructed when deterioration is already evident, then at least one seal, preferably a double seal, should be budgeted for to achieve a 15-year design life and a further reseal for a 20-year design life. Once top-down cracking becomes too severe or extensive, then milling off and replacing the wearing course may be a cheaper, or preferred, maintenance option.

Safety considerations

3.18 In developing countries safety considerations are not always given the priority they merit during the manufacture and construction of HMA wearing courses. To provide good skid resistance properties during wet weather a wearing course must have a good surface texture to prevent aquaplaning at high speed. The coarse aggregate should also have good resistance to polishing to reduce the probability of slow speed skidding.

4 Materials for HMA

4.1 It is essential that the properties of the component materials of HMA meet minimum standards to ensure the material has a satisfactory performance.

Aggregates

4.2 Aggregate is the major component in HMA and the quality and physical properties of this material has a large influence on mix performance. Typically the qualities required of aggregates are described in terms of shape, hardness, durability, cleanliness, bitumen affinity and porosity. In addition to these properties, the micro texture of the aggregate particles will also strongly influence the performance of a compacted HMA layer. Smooth-surfaced river gravel, even partly crushed, may not generate as much internal friction as a totally crushed aggregate with particles having a coarse micro texture.

4.3 The coarse aggregates used for making HMA should be produced by crushing sound, unweathered rock or natural gravel. Gravel should be crushed to produce at least two fractured faces on each particle.

4.4 The aggregate should have the following characteristics:

- be clean and free of clay and organic material;
- be angular and not excessively flaky, to provide good mechanical interlock;
- be strong enough to resist crushing during mixing and laying as well as in service;
- be resistant to abrasion and polishing when exposed to traffic;
- be non absorptive - highly absorptive aggregates are wasteful of bitumen and also give rise to problems in mix design; and
- have good affinity with bitumen - hydrophilic aggregates may be acceptable only where protection from water can be guaranteed, or a suitable adhesion agent is used.

4.5 Filler (material finer than 0.075 mm) can be crushed rock fines, Portland cement or hydrated lime. Portland cement or hydrated lime is often added to natural filler (1-2 per cent by mass of total mix) to improve the adhesion of the bitumen to the aggregate.

4.6 Filler has an important effect on voids content and the stiffness of the bitumen-fines matrix. The SG of the filler must be taken into account because, for instance, equal masses of Portland cement and fresh hydrated lime will have very different bulk volumes and, therefore, different effects on mix properties.

4.7 The required properties for aggregates are given in Table 4.1 and summaries of the relevant test methods are given in Appendix A.

4.8 In the UK detailed specifications have been developed for the Polished Stone Value required at sites which present different degrees of risk (Department of Transport, UK (1994). These specifications are reproduced in Table 4.2.

Bitumen for HMA

4.9 There are three important properties or characteristics of paving grade bitumens. These are consistency (usually called viscosity), purity and safety.

4.10 Traditionally, paving grade bitumens have been specified in term of their penetration, but the measurement of viscosity provides a more accurate method of specifying binder consistency and a more effective method of determining the temperature susceptibility of the bitumen. This allows the most appropriate mixing and compaction temperature for the asphalt mix to be established by using the Bitumen Test Data Chart (BTDC) developed by Heukelom (1969)(1973), which is illustrated in Appendix B.

4.11 Several authorities now produce alternative specifications based on viscosity. Suitable apparatus for measuring viscosity may not be readily available in developing countries and, therefore, both methods of specification are presented below. When ordering bulk bitumen supplies, it should be possible to obtain evidence of compliance with viscosity specifications since the necessary equipment will be available at the refinery.

Pre-hardening of bitumen

4.12 Bitumen samples should be tested in both the ‘as delivered state’ and also after pre-hardening, which is intended to simulate the ageing of a bitumen during storage, mixing and construction. Two test methods are used to pre-age bitumen, the Thin Film Oven Test (TFOT) and the Rolling Thin Film Oven Test (RTFOT). The RTFOT test is considered to be the best method of simulating the ageing of bitumen during the construction process but, again, this apparatus may not be readily available. The TFOT can be used for penetration graded specifications but, where possible, the RTFOT equipment and a viscosity graded specification should be used.

Requirements for penetration graded bitumens

4.13 The basic requirements for penetration graded bitumens are:

i Bitumen shall be prepared by the refining of bitumen obtained from crude oil by suitable methods. The bitumen shall be homogeneous and shall not foam when heated to 175°C.

ii The various grades of bitumen shall conform to the requirements in Table 4.3.

Requirements for viscosity graded bitumens

4.14 Authorities such as AASHTO, ASTM, the Standards Association of Australia (AS 2008, 1980) and the South African Bureau of Standards have produced specifications based on viscosity. The AASHTO and ASTM tests use capillary viscometers whilst the South African specifications utilise a rotary viscometer which is ideal for acquiring data to plot on the Bitumen Test Data Chart. The South Africa Bureau of Standard's requirements for bitumen viscosity are shown in Table 4.4.

European specifications for paving grade bitumens

4.15 The Comité Européen De Normalisation (CEN) has drawn up standards (EN 12591: 1999, or BS EN 12591:2000) for bitumen and bituminous binders which are now used as national standards in nineteen European countries. The CEN standards include country-specific variations in specifications and precision statements for test methods for bitumen and bituminous binders. The British Standard BS 3690-1:1989 is now obsolete.

4.16 Specifications for grades most appropriate for use in tropical countries have been selected from the standards and reproduced in Table 4.5. In principle, specifications can be selected that are suitable for use in hot countries, however, it is essential that authorities refer to original CEN standards before adopting any of the recommendations.

Bitumen durability

4.17 Bitumens derived from different sources of crude oil can have varying resistance to oxidation. Their characteristics can be further affected by the type of refining plant in which they are produced. The main purpose of most oil refining is to obtain the valuable distillates such as naphtha's, fuel and heavier oils. After distillation, the bitumen residue is usually too soft to be used for paving and must be treated further. There are two methods of treatment. The first involves either air blowing (or oxidation) of the residue, typically carried out in fuel producing refineries. The second is blending with propane-precipitated bitumen, which is a by-product of the manufacture of lubricating oil.

4.18 Depending upon the properties of the crude oil and the processing, bitumen produced in the propane-precipitation method can be more durable. This can be determined by the extended RTFOT developed in Australia (Dickinson, 1984). Financial restraints may mean that authorities must purchase bitumen at the most competitive open market prices. However, the import of more durable bitumen should be seriously considered for major projects such as international airfields and important transcontinental routes.

Table 4.1 Required properties for HMA aggregates

Property	Test		*Properties* Wearing course	Binder course
Cleanliness	Sand equivalent:[1] for < 4.75mm fraction			
		<1.5 x 10^6 esa	>35	
		>1.5 x 10^6 esa	>40	
	(Material passing 0.425mm sieve)			
	Plasticity index[2]		<4	
	Linear shrinkage %		<2	
Particle shape	Flakiness index[3]		<35	
Strength	Aggregate Crushing Value (ACV)[4]		<25	
	Aggregate Impact Value (AIV)[4]		<25	
	10% FACT (dry) kN[4]		>160	
	Los Angeles Abrasion (LAA)[5]		<30	<35
Abrasion	Aggregate Abrasion Value (AAV)[4]			
		250-1000 cv/lane/day	<16	–
		1000-2500 cv/lane/day	<14	–
		>2500 cv/lane/day	<12	–
Polishing	Polished Stone Value[4]		(see Table 4.2)	–
Water absorption	Water absorption[6]		<2	
Soundness[7] (5 cycles, % loss)	Sodium Sulphate Test:	Coarse	<10	
		Fine	<16	
	Magnesium Sulphate Test:	Coarse	<15	
		Fine	<20	
Bitumen affinity	Immersion Mechanical Test: Index of[8] retained Marshall stability		>75	
	Static Immersion Test[9]		>95% coating retained	
	Retained Indirect Tensile strength[10]		>79% (at 7%VIM)	

[1] *AASHTO T176*
[2] *British Standard 1377: Part 2*
[3] *British Standard 812, Part 105*
[4] *British Standard 812, Parts 110 to 114*
[5] *ASTM C131 and C535*
[6] *British Standard 812, Part 2*
[7] *AASHTO T104*
[8] *D Whiteoak (1990)*
[9] *AASHTO T182*
[10] *AASHTO T283*

Table 4.2 Minimum PSV for coarse surfacing aggregates for roads in Britain

<table>
<tr><th rowspan="2">Site definition</th><th colspan="14">Traffic (cv[1]/l/d) at end of design life</th></tr>
<tr><th>0 to 100</th><th>101 to 250</th><th>251 to 500</th><th>501 to 750</th><th>751 to 1000</th><th>1001 to 1250</th><th>1251 to 1500</th><th>1501 to 1750</th><th>1751 to 2000</th><th>2001 to 2250</th><th>2251 to 2500</th><th>2501 to 2750</th><th>2751 to 3250</th><th>Over 3250</th></tr>
<tr><td>1 Motorway (main line).
Dual carriageway (all purpose) non-event sections.
Dual carriageway (all purpose) minor junctions.</td><td colspan="8">55</td><td colspan="2">57</td><td colspan="2">60</td><td>65</td><td>68</td></tr>
<tr><td>2 Single carriageway non-event sections.
Single carriageway minor junctions.</td><td>45</td><td>50</td><td colspan="2">53</td><td>55</td><td colspan="2">57</td><td>60</td><td colspan="2">63</td><td colspan="2">65</td><td colspan="2">68</td></tr>
<tr><td>3 Approaches to and across major junctions (all limbs).
Gradient 5%-10%, longer than 50m (Dual downhill; single uphill and downhill).
Bend (not subject to 64 kph or lower speed limit) radius 100-250m.
Roundabout</td><td>50</td><td>55</td><td>57</td><td colspan="2">60</td><td colspan="2">63</td><td colspan="2">65</td><td colspan="2">68</td><td colspan="3">over 70</td></tr>
<tr><td>4 Gradient >10%, longer than 50m (Dual downhill; single uphill and downhill).
Bend (not subject to 64 kph or lower speed limit) radius <100m.</td><td>55</td><td>60</td><td colspan="2">63</td><td colspan="2">65</td><td colspan="2">68</td><td colspan="3">over 70</td><td colspan="3" rowspan="2"></td></tr>
<tr><td>5 Approach to roundabout, traffic signals, pedestrian crossing, railway level crossing, etc.</td><td>63</td><td>65</td><td colspan="2">68</td><td colspan="7">over 70</td></tr>
</table>

[1]*Commercial vehicles are defined as those over 15 kN unladen weight*

Table 4.3 Minimum requirements for penetration grade bitumen

Test		*Test method (ASTM)*	*Penetration grade*		
			40/50	*60/70*	*80/100*
Based on original bitumen					
Penetration at 25˚C		D 5	40-50	60-70	80-100
Softening point (˚C)		D 36	49-59	46-56	42-51
Flash point (˚C)	Min	D 92	232	232	219
Solubility in trichloroethylene (%)	Min	D 2042	99	99	99
TFOT heating for 5h at 163˚C		D 1754			
a. Loss by mass (%)	Max	–	0.5	0.5	0.8
b. Penetration (% of original)	Min	D 5	58	54	50
c. Ductility at 25˚C	Min	D 113	–	50	75

Table 4.4 South African specifications for viscosity graded bitumens (SABS, 1997)

Test		*Test method (various)*	*Penetration grade*		
			40/50	*60/70*	*80/100*
Based on original bitumen					
Penetration at 25°C		D 5	40-50	60-70	80-100
Softening Point (°C)		D 36	49-59	46-56	42-51
Viscosity 60°C (Pa.s)		D 4402	220-400	120-250	75-150
Viscosity 135°C (Pa.s)		D 4402	0.27-0.65	0.22-0.45	0.15-0.40
Ductility at 10°C (cm) [1]	Min	DIN 52013	–	–	100
Ductility at 15°C (cm) [1]	Min	DIN 52013	100	100	–
Tests on residue from RTFOT		D 2872			
a. Loss in mass (%)	Max		0.5	0.5	0.5
b. Viscosity at 60°C (% of original)	Max	D 4402	300	300	300
c. Ductility at 10°C (cm)	Min	DIN 52013	–	–	5
d. Ductility at 15°C (cm)	Min	DIN 52013	5	10	–
e. Softening point (°C)	Min	D 36	52	48	44
f. Increase in softening point (°C)	Max		9	9	9
g. Penetration (% of original)	Min		60	55	50
Spot test (% xylene)	Max	T 102	30	30	30

[1] *These specifications are under review.*

Table 4.5 Part of the European (CEN) specifications for paving grade bitumens

Test[1]			Unit	Grade designation						
				20/30	*30/45*	*35/50*	*40/60*	*50/70*	*70/100*	*100/150*
Penetration at 25°C	EN 1426		x 0.1mm	20-30	30-45	35-50	40-60	50-70	70-100	100-150
Softening point	EN1427		°C	55-63	52-60	50-58	48-56	46-54	43-51	39-47
Solubility	EN 12592	minimum	% total mass	99.0	99.0	99.0	99.0	99.0	99.0	99.0
Flash point	BS EN ISO 2592	minimum	°C	240	240	240	230	230	230	230
Resistance to hardening at 163°C[2] EN 12607-1 or EN12607-3										
Change in mass		maximum ±	%	0.5	0.5	0.5	0.5	0.5	0.8	0.8
Retained penetration		minimum	%	55	53	53	50	50	46	43
Softening point after hardening		minimum	°C	57	54	52	49	48	45	41

[1] *Additional tests and different specifications are applicable in a number of EU countries (see BS EN 12591:2000).*
[2] *Rolling Thin Film Oven Test to be used for resolving disputes*

5 Mix design for HMA

Introduction to mix design methods

5.1 Ideally the design of an HMA mix involves the following iterative process:

i establishing candidate mixes with satisfactory volumetric composition;

ii testing to confirm that the compacted mix has the required properties for the expected traffic; and, if necessary,

iii adjusting the mix composition and re-test until the design requirements are satisfied.

5.2 Mix design for AC surfacing materials in developing countries is commonly based on the recommendations given in the Asphalt Institute Manual Series, MS-2 (1994) and is carried out using the Marshall test procedure. This method employs impact compaction with the Marshall hammer to produce briquettes of different compositions. The briquettes are then tested to ensure that the mix criteria are appropriate for the design traffic. An outline of the method is given in Appendix C.

5.3 In MS-2, heavy traffic is defined as greater than 1 million esa and 75 blow Marshall compaction is recommended for the design of AC's which are expected to carry this amount of traffic. However, the basic requirement of the method is that the level of Marshall compaction used should produce a density in the design mix which is equal to that which will be produced in the road after secondary compaction under traffic. Unfortunately there is no method for determining what this level of compaction should be, other than from empirical knowledge, and it is therefore common practice to use 75 blow compaction for all design traffic loads in excess of 1 million esa.

5.4 The design traffic loading of 1 million esa is now being exceeded by ever increasing margins and it was the need for a more comprehensive design method that led to development of the Superpave™ method of developed during the Strategic Highway Research Program (SHRP) in the USA. Superpave™ is a trademark of the Strategic Highway Research Program.

5.5 The Superpave™ procedure involves careful selection and detailed testing of bitumen (Asphalt Institute Superpave Series No.1, 1997) and the use of a gyratory compactor for mix design (Asphalt Institute Superpave Series No.2, Third Edition, 2001). Mix design requires that volumetric requirements are met and that the design mix will have compaction characteristics that are related to the expected traffic loading. An outline of the Superpave™ method is given in Appendix D.

5.6 AUSTROADS have developed a provisional procedure based on the use of a gyratory compactor and performance tests similar to those developed in the UK, namely Dynamic Creep and Resilient Modulus tests. Requirements are specified for three levels of design traffic where heavy traffic is defined as more than 5 million esa.

5.7 An 'Interim guidelines for the design of hot-mix asphalt in South Africa' has been introduced, (The South African National Roads Agency, 2001). The importance of volumetric design and compaction characteristics are emphasised in the guide. A modified Marshall procedure allows continuous monitoring of mix density to ensure that the mix has desirable compaction characteristics. Gyratory compaction is also used to confirm that satisfactory VIM will be retained under heavy and very heavy traffic. Appropriate performance tests are used in place of Marshall stability and flow tests. These tests include Indirect Tensile Strength and Resilient Modulus (ASTM D4123), Dynamic creep, Four Point Bending Beam test, and Mix Permeability amongst others. Practitioners should refer to the document to obtain a complete understanding of the methodology.

5.8 Authorities in developing countries will often encounter difficulties with more complex test methods, including the initial cost of establishing suitable laboratories and the maintenance and calibration of the equipment. The use of gyratory compactors is to be encouraged but it is expected that the Marshall procedure will remain the principle method of mix design for AC mixes in many developing countries for several years. Whilst the use of performance tests of the types described in Appendix E would be useful for the design of HMA for heavy traffic, guidance is given in Chapter 6 on a simple procedure based on refusal density.

5.9 A typical example of the effect of different levels of Marshall compaction is given in Appendix F. A simple method of compaction to refusal density is described in Appendix G as a method of ensuring the retention of a minimum VIM in HMA mixes used on severe sites.

Volumetric design of HMA mixes

5.10 For convenience, mix components are blended in proportion by mass and expressed as percentages of the complete mix. *However, the controlling factor in the design of mixes for all traffic levels is the volume of each mix component.*

5.11 The volumetric design of a compacted HMA is affected by:

i the proportions of the different aggregates and filler;

ii the specific gravity of the different materials;

iii where porous aggregate is present, the amount of bitumen absorbed; and

iv the amount of non-absorbed bitumen.

The basic definitions used in volumetric design are summarised here. More detail is contained in Appendix C.

Air Voids (VIM) – the total volume of air, expressed as a percentage of the bulk volume of the compacted mixture, which is distributed throughout a compacted paving mixture and is located between the coated aggregate particles.

Effective Bitumen Content (P_{be}) – this governs the performance of the mix. It is the portion of bitumen that remains as a coating on the outside of the aggregate particles. Any bitumen that is absorbed into the aggregate particles does not play a part in the performance characteristics of the mix, but has the effect of changing the specific gravity of the aggregate.

Voids in the Mineral Aggregate (VMA) – the volume of void space between the aggregate particles of a compacted paving mixture. It is the sum of VIM and P_{be} expressed as a percent of the total volume of the sample.

Voids Filled with Bitumen (VFB) – the portion of the volume of void space between the aggregate particles (VMA) that is occupied by the effective bitumen.

The specific gravity of the mix components must also be determined. These are defined as:

Bulk Specific Gravity (G_{sb}) – the ratio of the weight in air of a unit volume of a permeable aggregate (including both permeable and impermeable voids within the aggregate particles) at a stated temperature to the weight in air of equal density of an equal volume of gas-free distilled water at a stated temperature (See Figure 5.1).

Apparent Specific Gravity (G_{sa}) – the ratio of the weight in air of a unit volume of an impermeable aggregate at a stated temperature to the weight in air of equal density of an equal volume of gas-free distilled water at a stated temperature (See Figure 5.1).

Effective Specific Gravity (G_{se}) **–** the ratio of the weight in air of a unit volume of a permeable aggregate (excluding voids permeable to bitumen) at a stated temperature to the weight in air of equal density of an equal volume of gas-free distilled water at a stated temperature (See Figure 5.1).

Specific Gravity of bitumen (G_b) **–** the ratio of the weight in air of a unit volume of bitumen at a stated temperature to the weight in air of equal density of an equal volume of gas-free distilled water at a stated temperature.

Maximum Specific Gravity of the loose mixed material (G_{mm}) – the ratio of the weight in air of a unit volume of uncompacted HMA at a stated temperature to the weight in air of equal density of an equal volume of gas-free distilled water at a stated temperature.

Bulk specific gravity (G_{mb}), of the compacted material **–** the ratio of the weight in air of a unit volume of compacted HMA at a stated temperature to the weight in air of equal density of an equal volume of gas-free distilled water at a stated temperature.

Nomenclature and test methods for volumetric design are summarised in Table 5.1.

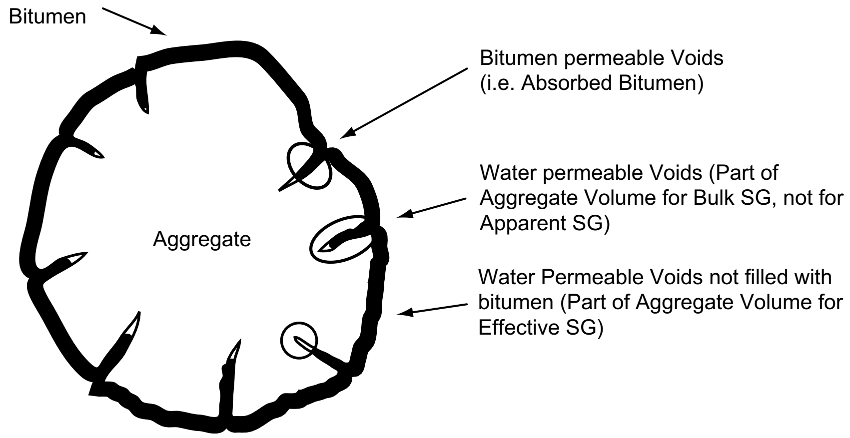


Figure 5.1 Illustrating the bulk, effective and apparent specific gravity; air voids; effective bitumen content in a compacted paving mix (Asphalt Institute, SP-2, 2001)

Table 5.1 Volumetric nomenclature and test methods

Component	*Volumetric description*	*Nomenclature*	*Test method*	
			ASTM	*AASHTO*
Constituents	Bulk Specific Gravity of coarse aggregate	G_{ca}	C127	T85
	Bulk Specific Gravity of fine aggregate	G_{fa}	C128	T84
	Bulk Specific Gravity of mineral filler	G_{f}	D854	T100
	Bulk Specific Gravity of total aggregate	G_{sb}	–	–
	Bulk Specific Gravity of bitumen	G_{b}	D70	T228
Mixed material	Bulk Specific Gravity of compacted material	G_{mb}	D2726	T166
	Maximum Specific Gravity of loose material	G_{mm}	D2041	T209
	Air voids	VIM	D3203	T269
	Effective bitumen content	P_{be}	–	–
	Voids in mineral aggregate	VMA	–	–
	Voids filled with bitumen	VFB	–	–

5.12 The representation of volumes in a compacted bituminous mixture are shown in Figure 5.2.

Summaries of test methods, requirements of test precision, calculations of volumetric parameters of VIM, P_{be}, VMA and VFB and a worked example are given in Appendix C.

Aggregate particle size distributions for HMA

5.13 There may be sufficient knowledge to justify the use of locally derived aggregate particle size distributions for HMA, otherwise the distributions given below should be considered.

5.14 Irrespective of the particle size distribution that is chosen, a blend of aggregate particles suitable for dense AC surfacings must produce a mix which will:

i have sufficient VMA to accommodate enough bitumen to make it workable during construction;

ii be durable in service; and

iii retain a minimum of 3 per cent VIM after secondary compaction by traffic.

5.15 An aggregate blend for HMA is characterised by:

i *the nominal maximum stone size* – defined as one sieve size larger than the first sieve to retain more than ten per cent; and

ii *the maximum stone size* – defined as one sieve size larger than the nominal maximum size.

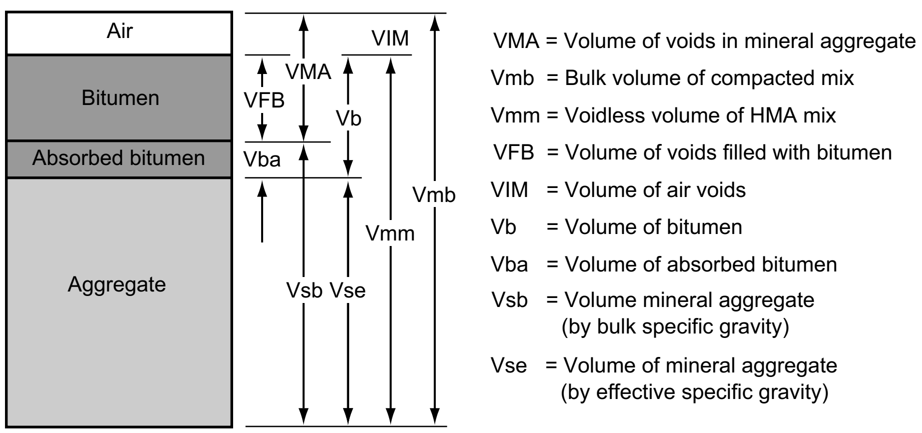


Figure 5.2 Representation of volumes in a compacted HMA specimen (Asphalt Institute, MS-2, 1994)

The nominal maximum stone size determines the minimum VMA required in the aggregate blend. The maximum stone size that can be used in a mix is governed by the proposed thickness of the HMA layer.

5.16 To achieve good compaction the layer thickness will normally have to be between twice the maximum stone size for fine mixes and four times the maximum stone size for mixes with a high content of coarse aggregates. Mixes normally recommended for severe traffic loading, or which fall below the Superpave™ restricted zone, would be in the latter category.

Particle size distributions for AC wearing courses

5.17 Authorities will often base the choice of particle size distribution on local experience, or the recommendations of the Asphalt Institute (SS-1, or MS-2, Table 2.1, 1994). Particle size distributions recommended by the Asphalt Institute for wearing course layers are shown in Table 5.2.

Table 5.2 Particle size distributions for AC wearing courses (Asphalt Institute, 1994)

	Nominal maximum stone size (mm) *Percentage passing sieve size*		
Sieve size (mm)	*19*	*12.5*	*9.5*
25	100		
19	90 – 100	100	
12.5	–	90 – 100	100
9.5	56 – 80		90 – 100
4.75	35 – 65	44 – 74	55 – 85
2.36	23 – 49	28 – 58	32 – 67
1.18	–	–	–
0.600	–	–	–
0.300	5 – 19	5 – 21	7 – 23
0.075	2 – 8	2 –10	2 – 10

5.18 Mix manufacture is done on the basis of blending materials by mass whilst mix design is done on a volumetric basis. If the specific gravity of the individual unblended aggregates differ by more than 0.19 (Asphalt Institute, MS-2, 1994) then the specified masses of the different aggregates in the blend must be adjusted so that the volumetric properties of the plant mix are correct.

5.19 A compacted blend of crushed aggregates will give a maximum density if the particle size distribution follows the Fuller curve. However, this minimises VMA and produces a mix which will be very sensitive to proportioning errors. It is best practice to modify the distribution away from the maximum density line.

5.20 The Superpave™ mix design procedure addresses the need for sufficient VMA by specifying control points within which the particle size distribution must fit and a restricted zone. VMA is increased both by displacing the particle size distribution away from the maximum density line and by avoiding the restricted zone. An example of a generalised aggregate grading chart showing control points and the restricted zone is shown in Figure 5.3. The maximum density grading is shown as a straight line where the sieve sizes on the x-axis have been raised to the power 0.45.

5.21 Use of the restricted zone is not compulsory. It was originally specified to limit the amount of natural rounded sand that could be used in a blend. A mix containing only fully crushed rock fines and having a particle size distribution which passes through the restricted zone may develop good particle interlock and sufficient VMA. It is recommended, therefore, that the restricted zone is not adopted as an essential requirement of local specifications, rather that it is an option for heavily trafficked roads.

5.22 The combined effect of VMA selection and particle size distribution becomes more sensitive as traffic loading increases, particularly under the severe conditions which apply in many developing countries. Particle size distributions which pass below the restricted zone will normally provide the most effective material for roads carrying very heavy traffic and for severe sites, but this must be confirmed by adequate laboratory testing. It is possible that adjustment of the proportions of larger sized aggregates will produce an equivalent increase in VMA as adjusting the particle size distribution to pass outside the restricted zone.

5.23 The specified control points and restricted zones for HMA wearing course mixes depend on the nominal maximum stone size. Superpave™ particle size distributions and an example of a complete chart is given in Appendix D.

Particle size distributions for AC binder courses and roadbases

5.24 The Asphalt Institute MS-2 (1994) and Superpave™ (2001) do not describe particle size distributions specifically for binder courses and roadbases. In practice, asphalt surfacings thicker than about 70mm are laid as two layers and the relationship between the thickness of a layer and the maximum stone size largely determines the particle size distribution that will be used (see paragraph 5.16).

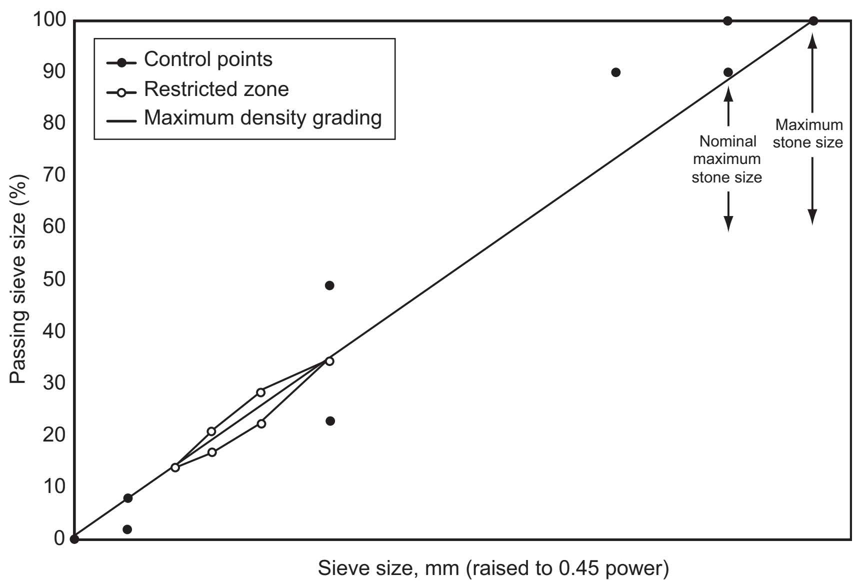


Figure 5.3 Example of generalised Superpave™ particle size distribution

5.25 Some of the particle size distributions recommended by the Asphalt Institute that are suitable for binder courses and roadbases are shown in Table 5.3. Because the Marshall test method cannot be used to design mixes with aggregate larger than 25mm, the design of coarse binder courses and roadbases tends to rely on empirical knowledge.

Table 5.3 Particle size distributions for AC roadbases and binder courses (Asphalt Institute, 1994)

	Nominal maximum stone size (mm) *Per cent passing sieve size*	
Layer	*Roadbase*	*Binder course*
Sieve size (mm)	*37.5*	*25*
50	100	
37.5	90 – 100	100
25	–	90 – 100
19	56 – 80	–
12.5	–	56 – 80
4.75	23 – 53	29 – 59
2.36	15 – 41	19 – 45
0.300	4 – 16	5 – 17
0.075	0 – 6	1 - 7

Particle size distributions for Dense Bitumen Macadam (DBM)

5.26 Particle size distributions recommended for DBM wearing courses and for binder course and roadbase layers (BSI:1993) are shown in Tables 5.4 and 5.5. These mixes have traditionally been made to recipes, but the wearing course mixes could be designed by the Marshall method. If the available aggregates are known to give good results when used in AC mixes it can be expected that DBM mixes using similar aggregate will be satisfactory for traffic loading up to 1 million esa. An important advantage in using DBM mixes is that it is easier to carry out detailed mix control during production. Only determinations of the particle size distribution of the aggregate delivered by the cold feed system, mix composition and the density of the compacted mat need be made.

Table 5.4 Particle size distributions for DBM wearing courses

	UK nomenclature for size (mm)	
	Percentage passing sieve size	
Sieve size (mm)	*14*	*10*
20	100	
14	95 – 100	100
10	70 – 90	95 – 100
6.3	45 – 65	55 – 75
3.35	30 – 45	30 – 45
1.18	15 – 30	15 – 30
0.075	3 – 8	3 – 8
Typical bitumen content (%)	4.9	5.2

Table 5.5 Particle size distributions for DBM binder course and roadbase layers

	UK nomenclature for size (mm)		
	Percentage passing sieve size		
	Binder course	*Roadbase*	
Sieve size (mm)	*20*	*40*	*28*
50	–	100	–
37.5	–	95 – 100	100
28	100	70 – 94	90 – 100
20	95 – 100	–	71 – 95
14	65 – 85	56 – 76	58 – 82
10	52 – 72	–	–
6.3	39 – 55	44 – 60	44 – 60
3.35	32 – 46	32 – 46	32 – 46
0.300	7 – 21	7 – 21	7 – 21
0.075	2 – 9	2 – 9	2 – 9
Typical bitumen content (%)	4.7	3.5	4.0

6 Mix design specifications

Mix design for continuously graded wearing courses

6.1 AC wearing courses tend to be sensitive to variations in composition. A high level of quality control is essential during laboratory design, manufacture, compliance testing and construction. The small range of VIM values shown in Table 6.1 and the effect they have on mix performance illustrates this sensitivity.

Table 6.1 Critical values of VIM in a wearing course

VIM (per cent)	*Effect*
>5	Increasingly permeable to air and prone to oxidation of bitumen.
4 or 5	Target for design.
3-5	For a durable and stable mix.
<3	Prone to plastic deformation under heavy loading.

6.2 In order to achieve a balance of mix properties it is important that the aggregate structure of an HMA has sufficient VMA. The minimum VMA required is related to the nominal stone size as shown in Table 6.2.

Table 6.2 Minimum VMA specified for AC mixes

Nominal maximum stone size (mm)	*Minimum VMA (per cent) @ Design VIM (per cent)*	
	4.0	*5.0*
37.5	11.0	12.0
25	12.0	13.0
19	13.0	14.0
12.5	14.0	15.0
9.5	15.0	16.0

VMA and bitumen film thickness

6.3 Whilst VMA is crucial to the correct volumetric design of HMA it is important to be aware of the possible limitations in rigidly specifying values and also of the difficulties in accurately measuring VMA. The variation in bitumen content with change in maximum stone size should actually be related to the surface area of all of the aggregate particles in an HMA. However, most authorities will not be able to determine absolute values for surface area and will rely on determinations of VMA.

6.4 Unfortunately the measurement of VMA is subject to large variability (Hinrichsen and Heggen, 1996) with typical standard deviations of 1.3 per cent. This is a large value in comparison to the incremental steps given in normal specifications (see Table 6.2). In addition, two particle size distributions having different maximum sized aggregate may overlap to a considerable degree and the difference in aggregate surface area may not be sufficiently large to warrant a large change in the specified VMA.

6.5 It is recommended that bitumen film thickness (i.e. the nominal thickness of non-absorbed bitumen coating the aggregate particles) is calculated, as shown in Appendix C, and used to assist in the design process. If the bitumen film thickness is less than 7 microns it is recommended that the determination of VMA be reviewed. Finally, evidence from field compaction trials, including the volumetric properties of cores cut from the trials, will help to confirm that the mix is sufficiently workable and that specified properties are obtained.

6.6 It is recommended that AC mix design using the Marshall method be based on three categories of design traffic:

i < 5 million esa;

ii > 5 million esa; and

iii severe sites - defined as climbing lanes and junctions that are subject to slow moving heavy traffic.

For design traffic less than 5 million esa

6.7 In principle, any of the wearing course or binder course gradings described in Chapter 5 can be used as a running surface for traffic loading up to 5 million esa. The larger stone mixes have to be placed in thicker layers and the surface finish of such mixes would have a coarser texture. All mixes should be designed to the Asphalt Institute (MS-2, 1994) Marshall criteria for wearing courses shown in Table 6.3. It will be noted that a single value of 4 per cent VIM at the optimum bitumen content is recommended.

6.8 Some variation in mix composition is to be expected during plant manufacture and MS-2 recommends that variations in bitumen content should be restricted to produce a variation in VIM of only 4 ± 0.5 per cent at the design level of Marshall compaction.

For design traffic greater than 5 million esa

6.9 Local experience may justify the use of similar design criteria to those shown in Table 6.3 for design traffic greater than 5 million esa. However, such

Table 6.3 AC wearing course specifications for up to 5 million esa

Category and design traffic (million esa)	*Number of blows of Marshall compaction hammer*	*Minimum Stability (N)*	*Flow (mm)*	*VFB (%)*	*VIM at optimum bitumen content (%)*
Heavy (1 - 5)	75	8000	2-3.5	65-75	4
Medium (0.4 – 1)	50	5300	2-4.0	65-78	4
Light (<0.4)	35	3300	2-4.5	70-80	4

increases must be based on sound data. In particular:

i aggregate sources which are known to produce very stable mixes should be used; and

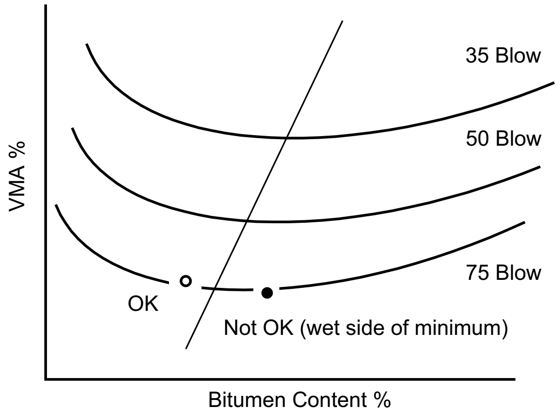
ii where applicable, care must be taken to classify correctly lengths of road as 'severe sites' (See paragraph 6.6) and to design a suitable mix for these locations.

6.10 Where there is no extensive local experience of the performance of HMA then the Marshall requirements for mixes designed for traffic greater than 5 million esa are summarised in Table 6.4.

Other considerations for design of continuously graded mixes

6.11 When a given aggregate blend is compacted in the Marshall test, VMA decreases as the bitumen content is increased until a minimum value of VMA is obtained. Thereafter, more bitumen causes an increase in VMA, indicating that the aggregate structure is becoming overfilled with bitumen, and will result in the mix being susceptible to plastic deformation. *It is important* therefore, that the design bitumen content is slightly less than that which gives the minimum VMA at the selected compaction level, as shown in Figure 6.1.

6.12 In the refusal density test (see paragraph 5.9 and Appendix G) VMA tends to remain constant until the structure starts to become overfilled.



Note: the minimum VMA values must satisfy the requirements given in Table 6.2

Figure 6.1 Effect of Marshall compactive effort on VMA and VIM (Asphalt Institute, 1994)

Mix design for severe sites

6.13 Without sufficient knowledge of the degree of secondary compaction that will occur on severe sites any selection of a level of Marshall compaction becomes arbitrary. In comparison, compaction to refusal provides a 'reference density' because the aggregate structure cannot be compacted any further. Particle size distributions can, therefore, be selected to give VMA that will accommodate sufficient bitumen to ensure good workability during construction and retain a minimum of 3 per cent VIM at refusal density. However, it is important that a

Table 6.4 AC wearing course specification for more than 5 million esa

Category and design traffic (million esa)	*Number of blows of Marshall compaction hammer*	*Minimum stability (N)*	*Flow (mm)*	*VFB (%)*	*VIM at optimum bitumen content (%)*
Very heavy (>5)	75	9000	2-3.5	65-73	5

compromise is reached between high VMA to accommodate enough bitumen to make the mix workable and sufficient fines to provide a strong mix. It is also important that the coarse aggregate is strong enough to withstand vibratory compaction without significant breakdown of the particles.

6.14 Dense wearing course mixes with low VMA will not be suitable for this type of surfacing because the design bitumen content will be too low for the mix to be workable. Suitable particle size distributions will be of the binder course type and the particle size distribution will probably pass beneath the relevant Superpave™ restricted zone (see Appendix D). Suitable particle size distributions are given in Tables 5.3 and 5.5, which allow a maximum particle size of up to 37.5mm. A Marshall design should be carried out on the selected mix but with no aggregate larger than 25mm. The Marshall design must meet the requirements for stability and flow given in Table 6.4 for very heavy traffic.

6.15 If the Marshall requirements are satisfied then coarse aggregate between 25mm and 37.5mm or 25mm and 28mm, depending upon the particle size distribution selected, may be included in the final mix if desired. This will provide a better balance between maximum particle size and the thickness of the layer to be constructed (see paragraph 5.16). The additional coarse aggregate should be from the same source as the aggregate used in the Marshall design.

6.16 The particle size distribution given in Table 5.3 allows up to 10 per cent of aggregate particles between 25mm and 37.5mm. However, restricting this to a maximum of 5 per cent may also result in less of a problem with segregation, which can be evaluated during pre-construction compaction trials. A binder course aggregate grading having a maximum particle size of 28mm and complying with Table 5.5 will often be a good compromise.

6.17 It is recommended that HMA designed to refusal density is laid to a compacted thickness of 2.5 to 4 times the maximum aggregate particle size to obtain satisfactory workability. The layer thickness can, therefore, range from 70mm to more than 100mm for particle size distributions complying with Tables 5.3 and 5.5.

6.18 Compaction to refusal could be achieved in the laboratory by applying several hundred blows of the Marshall hammer to each face of the test briquettes, but this is not practical. The preferred method is to use an electric vibrating hammer which is more representative of field compaction, and is a much quicker operation. The test method is based on the Percentage Refusal Density (PRD) test (BSI, 1989), (see Appendix G) which is being incorporated into a CEN Standard, prEN 12697-9, Test methods - Reference density).

6.19 The test moulds for this method are large enough to allow the design of mixes containing aggregate particles larger than 25mm. The apparatus is easily transportable and can be used to compact hot mix samples anywhere on site provided a suitable power source is available.

6.20 The design bitumen content is determined by compacting samples to refusal using the method described in Appendix G. The thickness of the compacted samples should be approximately the same as the compacted layer to be laid on the road.

6.21 The mix must be workable at the design bitumen content. If necessary, the particle size distribution must be adjusted until VMA is high enough to accommodate sufficient bitumen. A minimum calculated bitumen film thickness (see Appendix C) of 7 to 8 microns has been found to be a good indicator of a workable mix. However, the overriding requirement is that at refusal density VIM is 3 per cent. Pre-construction compaction trials are essential to the selection of the final mix design (see Appendix G).

Selection of grade of bitumen

6.22 60/70 penetration grade bitumen is generally recommended for use in HMA in hot climates. For severe sites the additional mix stiffness that should result from use of 40/50 penetration grade bitumen may be justified. Typically an increase in mixing temperature of up to 10°C will be necessary if the harder bitumen is used.

Use of recycled asphalt

6.23 The most satisfactory HMA which contain substantial amounts of recycled asphalt pavement (RAP) are likely to be binder course or roadbase mixes. The fundamental requirements for mix design are the same as for a mix containing entirely fresh material. Problems that will arise when RAP is incorporated into an HMA will be mainly associated with the quality and characteristics of this material. Guidance on the use of RAP is given in Appendix H.

7 Mix production

General requirements

7.1 Initial HMA design is often carried out to enable suitable aggregates to be selected before they are stockpiled for full-scale production. Stockpiling and calibration of the aggregate cold feed bins is then completed before a new mix design is made, using aggregate which has passed through the fully-operating asphalt plant.

7.2 The following are some of the factors that are important to the production of a consistent mix of good quality:

i building stockpiles with uniform distributions of aggregate sizes;

ii calibration of plant weigh scales;

iii calibration of gate settings on aggregate cold feed bins; and

iv correct adjustment of dust extraction equipment.

7.3 Having confirmed that a suitable laboratory design mix, which can be called the 'preliminary job mix formula', can be produced using the available aggregate sources, stockpiles of each material are then built. The quality and consistency of the stockpiled materials must be carefully monitored and the stockpiles constructed so as to minimise segregation. 'Dry' runs of aggregates are required to adjust cold bin vibrators and to calibrate gate settings. The rate of flow of sand-sized fines through the gate of a cold bin can be seriously affected if the moisture content of the material changes. Stockpiles of fine materials may therefore need to be covered to prevent frequent changes in flow characteristics. The use of efficient, variable-control, vibration devices on the cold feed bins to maintain steady flows of materials is important. Even then fines may still 'bridge' in the cold feed bin and it will be necessary to manually break down the material to maintain a steady rate of flow.

7.4 It may be found that fines collected through the plant's cyclone system do not have desirable properties, or are in excess of requirements. Also cement or hydrated lime may be required as an anti-stripping agent and the natural filler content may have to be reduced to allow for this. Thus for the production of good quality AC wearing course material it is important to have a separate filler feed and weighing system on batch plants so that the volumes of these materials can be controlled.

7.5 Once the settings for the cold aggregate feed have been made to produce the required blend, the material should be run through the fully-operating asphalt plant without the addition of bitumen. Mix design is then repeated using the plant-run aggregate and added filler where this is appropriate. If necessary, adjustments are made until a suitable mix design is produced. This mix is likely to be slightly different to the preliminary job mix formula and can be called the 'trial job mix formula'.

7.6 Trial mixes are then made in the asphalt plant with the addition of bitumen and filler in the pre-determined proportions. This plant mix must be tested to ensure that volumetric and Marshall design requirements are satisfied. If necessary, further adjustments to the mix proportions should be made and, in exceptional cases, the need to obtain different aggregates must be considered if the required mix specifications cannot be met with the existing materials.

7.7 Having established a plant mix design (it can be called the 'job mix formula') tolerances must be applied to the composition of the plant produced HMA. It is important that the required Marshall and volumetric criteria are met over the range of permitted tolerances. The control of variations in mix composition with respect to design criteria are discussed in Appendix C and typical plant mix tolerances are summarised in Table 7.1.

Table 7.1 Tolerances for the manufacture of AC

Tolerances for mix constituents			
		Bitumen content (%)	
Passing sieve size (mm)	*Permitted range (%)*	*Wearing course*	*Binder course*
>12.5	±8		
9.5	±7		
4.75	±7	±0.3	±0.5
2.36	±6		
300 microns	±5		
75 microns	±3		

7.8 The tolerances for the aggregate grading are for a single test result and are applied to the job mix formula to establish a particle size distribution envelope with which the plant mix must conform. It is expected that the new envelope will run approximately parallel with the boundaries of the original envelope, but it may overlap it.

7.9 As explained in paragraph 6.8 the tolerance on bitumen content should be reconciled with the very important recommendation that VIM at the design level of compaction should be within ± 0.5 per cent of the target value of either 4 or 5 per cent.

7.10 After a plant mix design is established, full-scale compaction trials must be carried out to confirm that the mix is workable and to determine the optimum use of rollers to achieve the required field densities.

Aggregate stock piles and cold feeds

7.11 The importance of good stockpile management and control of cold bin settings cannot be overstated. The quality and consistency of the HMA produced in the plant will be controlled by the uniformity of the stockpiled materials and by their correct proportioning from the aggregate cold bins, even for batch plants with separate aggregate hot bins. This is because a 5mm screen is often the smallest size used in these plants and AC wearing course mixes may contain 50 per cent of material finer than 5mm. Where these fines come from multiple sources such as crushed rock fines, natural sand and material adhering to the larger aggregate particles, it will only be possible to control the proportioning at the cold feed.

7.12 Once a plant mix specification has been established and production is started it is important that new materials for stockpiling are tested frequently to confirm that no significant changes in aggregate properties are occurring. It is advisable to create new stockpiles, rather than add to those which have been tested and are in use. If the properties of new aggregates cause the plant mix to fall outside of agreed specifications, and where this cannot easily be corrected, then a new mix design and plant-mix verification tests must be carried out.

7.13 Representative samples of the daily plant mix production and of stockpiled material must be taken to confirm compliance with all mix specifications. Advice on sampling frequency can be obtained from the Asphalt Institute Manual, MS-22 (Asphalt Institute, 2000).

8 Construction of asphalt surfacings

8.1 The purpose of this Chapter is to highlight some of the important aspects of construction of HMA surfacings. The Asphalt Institute Manual, MS-22 (Asphalt Institute, 2000) describes the principles of constructing hot mix asphalt pavements and the reader is encouraged to refer to this, or other publications (e.g. Hunter, 1994), if comprehensive detail of construction methods is required.

Mixing and compaction

8.2 Close control of mixing temperature is essential. In order to minimise the hardening of the bitumen, the lowest temperature commensurate with good coating of the aggregate and compaction requirements should be used and temperature variations should be minimised. This also makes it easier to obtain uniform compaction and to meet target densities.

8.3 Typical mixing temperatures are summarised in Table 8.1 for guidance. Where possible the viscosity of the bitumen should be measured over a range of temperatures and plotted on the Bitumen Test Data Chart (see Appendix B) so that the ideal mixing temperature, at which the viscosity of the bitumen is between approximately 0.2 and 0.5 Pa.s, can then be read from the chart.

Table 8.1 HMA mixing temperatures

Bitumen penetration grade	*Typical mixing temperature °C*
80/100	130-165
60/70	140-170
40/50	150-180

8.4 Thorough compaction during construction is vital because traffic is likely to give very little additional compaction outside of the wheelpaths. As the demand grows for HMA surfacings to carry higher design traffic loads, so the need for mixes to have higher resistance to rutting increases. This in turn demands higher compactive effort during construction.

8.5 Dead-weight steel rollers are effective and are essential for finishing joints between adjacent lanes and at the start of each laying operation. Pneumatic-tyred rollers are effective at compacting the lower part of thicker layers but, for high capacity roads, it may be necessary to use a vibratory roller to achieve the required densities. A number of roller types should be available and it is essential that pre-construction trials are carried out to determine the best combination of rollers, numbers of passes and any limitations which must be applied to rolling temperatures.

8.6 Vibrating rollers can cause considerable damage to an HMA layer if they are used incorrectly. For example, a vibrating roller operated at high forward speed with high amplitude compaction will leave a series of ridges in the mat. The optimum settings for the frequency and amplitude of vibration for the vibrating roller and the temperature range over which it can effectively be used for compaction must be determined.

8.7 In some cases it will be found that dead-weight rollers and pneumatic-tyred rollers are effective for completing the bulk of the compaction work, but a vibrating roller may be needed to ensure that the desired level of compaction is achieved. As few as 2 to 4 passes of a vibrating roller can be effective when used at the appropriate compaction temperature.

VIM after construction

AC mixes designed by the Marshall method

8.8 Control of compaction during construction is achieved by expressing *in situ* core densities relative to the density of plant mixed material compacted in the laboratory. The same number of blows of the Marshall hammer are applied to the plant mix as were used for the original laboratory mix design. Typically, the minimum level of relative compaction specified is 96 per cent.

8.9 Because an AC surfacing mix is designed to have 4 per cent VIM at the appropriate level of laboratory compaction, a relative density of 96 per cent will give *in situ* VIM of approximately 8 per cent (or 9 per cent if the design VIM is 5 per cent).

8.10 With VIM of more than 8 per cent the surfacing will be vulnerable to premature deterioration through ageing of the bitumen. Improved durability would be obtained by specifying a higher level of compaction, however, it is often difficult to consistently achieve relative compaction as high as 98 per cent and it is therefore recommended that the specified level of field compaction should be 97 per cent of laboratory density.

8.11 Where traffic is channelled only limited compaction will take place on areas outside the wheelpaths. This may result in the need for early maintenance if damage resulting from 'top-down' cracking is to be prevented. In any event the surface should be sealed when cracking develops and well before cracks penetrate to the full depth of the surfacing.

HMA designed by refusal compaction

8.12 For sites categorised as severe, a mix designed to 3 per cent VIM at refusal density and compacted during construction to a *mean* density of 95 per cent of refusal density will have 8 per cent VIM. The minimum specified density is normally 93 per cent of refusal density and, therefore, approximately 50 per cent of the constructed layer may have VIM of between 8 and 10 per cent. Because this type of mix cannot be compacted to less than 3 per cent there is every advantage in achieving as high a density as possible and careful use of vibrating rollers may consistently achieve densities in excess of 95 per cent of the design density.

8.13 Mixes of this type should be very resistant to long term secondary compaction under traffic and virtually no densification will occur outside of the wheelpaths. *It is essential* to seal these surfacings as part of the construction process to prevent ingress of water and premature 'top down' cracking (see Chapter 3). Sealing should be carried out as soon as surface hardness tests (TRL, 2000) show that there is sufficient resistance to chipping embedment. Cape seals and coarse textured slurry seals can be used as alternatives to surface dressings.

Segregation

8.14 In recent years TRL research has shown that segregation, i.e. where large aggregate in the HMA separates from the fines, is a common problem. In particular, segregation sometimes occurs at the point of delivery of HMA to the paver with the result that areas of surfacing with high values of VIM, and short life expectancy, occur at regular intervals of typically 30m along the road.

8.15 There is potential for segregation to develop every time graded aggregate or HMA is moved and this can occur, or its severity increased, by:

- unnecessary movement of materials in the stockpiles;
- unnecessarily high drop heights from pug mills or hot storage bins;
- letting storage bins, whether of cold or hot materials, run too low; and
- poor paving practice.

8.16 It is good practice to take material from a near vertical side of a stockpile and to avoid the use of machinery on top of the pile. 'Tidying up' of stockpiles can also lead to segregation. Reducing drop heights as far as is practicable may help to prevent large aggregate particles from running to the edge of lorry bodies each time a batch is dropped from a pug mill. When storage bins are run low the risk of segregation is increased.

8.17 The hopper in the paver must never be allowed to run low or to empty. If this occurs then segregated coarse aggregate from the back of the delivery lorry may be added to similar material from the front of the previous lorry. If the wings of the paver are then emptied, additional coarse material from the sides of the delivery lorry may also be added and a considerable excess of coarse aggregate delivered to the paver augers.

8.18 The paver augers will not re-mix segregated material. Indeed, incorrect setting up of the auger in relation to the screed, or the incorrect speed of operation of the auger, may also cause some segregation. It is good practice to keep the paver hopper well charged between each truck delivery.

8.19 Consideration should also be given to leaving the paver wings open during the day's work and discarding material left in the wings at the end of the day. If the wings are emptied then care must be taken to ensure that this does not result in the accumulation of segregated material.

8.20 It is very important that delivery of HMA to the paver is continuous, or at least that any delay between loads is short. If segregation is still apparent in the finished surfacing, indicated on the surface by a rougher texture, then the source of the problem must be identified and corrected. Back-casting with fines will not correct the problem and it may be necessary to design a new mix, perhaps with a reduced maximum stone size.

8.21 This chapter has summarised some of the key issues for constructing reliable HMA surfacings. For a more detailed description of construction operations the reader is referred to the manuals produced by the Asphalt Institute.

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11 Some of the AASHTO documents relevant to Superpave™

PP2 Standard Practice for Short and Long term Ageing of Hot Mix Asphalt.

TP-4 Standard Method for Preparing and Determining the Density of Hot Mix Asphalt Specimens by Means of SHRP Gyratory Compactor.

PP6 Guide for Grading or Verifying the Performance grade of an Asphalt Binder.

PPX Selection of Asphalt Binders (being developed).

M20 Specification for Penetration Graded Asphalt Cement.

M226 Specification for Viscosity Graded Asphalt Cement.

PP1 Practice for Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV)

T179 Test Method for Effect of Heat and Air on Asphalt Materials (Thin Film Oven Test).

T240 Test Method for Effect of Heat and Air on a Moving Film of Asphalt (Rolling Thin Film Oven Test).

TP1 Test Method for Determining the Flexural Creep Stiffness of Aspjhalt Binder Using the Bending Beam Rheometer (BBR).

TP3 Test Method for Determining the Fracture Properties of Asphalt Binder in Direct Tension (DT).

TP5 Test Method for Determining Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR).

12 Applicable British Standards for HMA

British Standards are being replaced by harmonised European standards under The European Committee for Normalisation, or Comité Européen de Normalisation (CEN). However, overseas authorities may have incorporated previous British Standards into their own design guides and much of the research work that forms the basis of this Road Note has used British Standards as references. A list of provisional CEN standards is given in Chapter 13.

The British Standards Institution is the independent national body for the preparation of British Standards. Enquiries should be addressed to the BSI, Linford Wood, Milton Keynes, MK14 6LE.

BS594 Hot rolled asphalt for roads and other paved areas.

Part 1 (1992)	Constituent materials and asphalt mixtures.
Part 2 (1992)	Transport, laying and compaction of rolled asphalt.

BS598 Sampling and examination of bituminous mixtures for roads and other paved areas.

Part 2 (1974)	Methods for analytical testing.
Part 3 (1985)	Methods for design and physical testing.
Part 100 (1987)	Methods for sampling for analysis.
Part 101 (1987)	Methods for preparatory treatment of samples for analysis.
Part 102 (1989)	Analytical test methods.
Part 104 (1989)	Sampling and examination of bituminous mixtures for roads and other paved areas.
Part 110 (1998)	Methods of test for the determination of wheel-tracking rate.

BS812 Sampling and testing of mineral aggregates, sands and fillers.

Part 1 (1975)	Methods of determining particle size and shape.
Part 2 (1975)	Physical properties.
Part 100 (1990)	General requirements for apparatus and calibration.
Part 101 (1984)	Guide to sampling and test procedures.
Part 102 (1989)	Methods of sampling.
Part 103 (1989)	Methods for determination of particle size distribution.
	Section 103.2 (1989) Sedimentation test.
Part 105	Methods for determination of particle shape.
	Section 105.1 (1989) Flakiness index.
Part 109 (1990)	Methods of determination of moisture content.
Part 110 (1990)	Method for determination of aggregate crushing value (ACV).
Part 111 (1990)	Methods for determination of ten per cent fines value (TFV).
Part 112 (1990)	Methods for determination of aggregate impact value (AIV).
Part 113 (1990)	Method for determination of aggregate abrasion value.
Part 114 (1989)	Method for determination of the polished-stone value.
Part 121 (1989)	Method for determination of soundness.

BS1377 Methods of test for soils for civil engineering purposes.

Part 2 (1990)	Classification tests.
BS 2000	Methods for test for petroleum and its products.
Part 397 (1995)	Recovery of bitumen binders - Dichloromethane extraction rotary film evaporator method.
Part 49 (1993)	Determination of needle penetration of bituminous material.
Part 58 (1993)	Determination of softening point of bitumen - Ring and ball method.

BS 3690 Bitumens for building and civil engineering.

Part 1 (1989)	Specifications for bitumens for roads and other paved areas. BS 3690: Part 1: 1989.

BS 4987 Coated macadam for roads and other paved areas.

Part 1 (1993)	Specifications for constituent materials and for mixtures.
Part 2 (1993)	Transport, laying and compaction.

BS Drafts for development

DD 213 (1993)	Determination of the indirect tensile stiffness modulus of bituminous mixtures.
DD 226 (1996)	Method for determining resistance to permanent deformation of bituminous mixtures subject to unconfined dynamic loading.

13 Applicable CEN standards for HMA

European national standards will eventually be replaced by harmonised standards produced by The European Committee for Normalisation, or Comité Européen de Normalisation (CEN). The final standards will be published by each national standardisation committee, such as the British Standards Institution, as their national Standards. It is expected that the various specifications and test methods for materials and design of asphalt will be published in December 2003. A list of some of the standards is given below.

Applicable CEN Standards

	Aggregates
prEN 13043	Aggregates for bituminous mixtures and surface dressings for roads and other trafficked areas
BS EN 932-1	Methods of sampling
BS EN 932-2	Methods for reducing laboratory samples
BS EN 932-5	Common equipment and calibration
BS EN 932-6	Definitions of repeatability and reproducibility
	Aggregates - Tests for geometric properties of aggregates
BS EN 933-1	Determination of particle size distribution - Sieving method
BS EN 933-2	Determination of particle size - Test sieves, nominal size of apertures
BS EN 933-3	Determination of particle shape - Flakiness index
BS EN 933-4	Determination of particle shape - Shape index
BS EN 933-5	Percentage of crushed and broken surfaces in coarse aggregate
BS EN 933-6	Flow coefficient of coarse aggregate
BS EN 933-8	Sand equivalent test
	Aggregates - Tests for mechanical and physical properties of aggregates
BS EN 1097-1	Determination of resistance to wear (Micro-Duval)
BS EN 1097-2	Methods for the determination of resistance to fragmentation
BS EN 1097-6	Determination of particle density and water absorption
BS EN 1097-7	Determination of the particle density of filler - Pyknometer method
BS EN 1097-8	Determination of the polished stone value
BS EN 1367-2	Magnesium sulfate test
	Aggregates - Tests for fillers
BS EN 1744-4	Water susceptibility of fillers for bituminous mixtures
BS EN 13179-1	Delta ring and ball test
BS EN 13179-2	Bitumen number
	Bitumen and bituminous binders
BS EN 12591	Specification for paving-grade bitumens
prEN 13924	Specification for hard paving-grade bitumens
prEN 14023	Specification for polymer modified bitumens
BS EN 58	Sampling of bituminous binders

Applicable CEN Standards (continued)

BS EN 1426	Determination of needle penetration
BS EN 1427	Determination of softening point - Ring and Ball method
BS EN ISO 2592	Determination of flash and fire points
BS EN 12592	Determination of solubility
BS EN 12593	Determination of Fraass breaking point
BS EN 12594	Preparation of test samples
BS EN 12595	Determination of kinematic viscosity
BS EN 12596	Determination of dynamic viscosity by vacuum capillary
BS EN 12607-1	Determination of the resistance to hardening under the influence of heat and air - RTFOT method
BS EN 12607-2	Determination of the resistance to hardening under the influence of heat and air - TFOT method
BS EN 12607-3	Determination of the resistance to hardening under the influence of heat and air - RFT method
prEN 13302	Determination of viscosity of bitumen using a rotating spindle apparatus
	Bituminous mixtures
prEN 13108-1	Material specification - Asphalt concrete
prEN 13108-2	Material specification - Asphalt concrete for very thin layers
prEN 13108-4	Material specification - Hot rolled asphalt
prEN13108-20	Quality - Type testing of asphalt mixes
BS EN 12697-1	Test methods - Soluble binder content
prEN 12697-2	Test methods - Particle size distribution
BS EN 12697-3	Test methods - Bitumen recovery, rotary evaporator
prEN 12697-5	Test methods - Maximum density
prEN 12697-6	Test methods - Bulk density, measurement
prEN 12697-8	Test methods - Air voids content
prEN 12697-9	Test methods - Reference density
prEN 12697-10	Test methods - Compatibility
prEN 12697-11	Test methods - Affinity between aggregate and binder
prEN 12697-12	Test methods - Moisture sensitivity
prEN 12697-15	Test methods - Segregation sensitivity
prEN 12697-19	Test methods - Permeability of porous asphalt specimen
prEN 12697-22	Test methods - Wheel tracking
prEN 12697-23	Test methods - Indirect tensile test
prEN 12697-26	Test methods - Stiffness
prEN 12697-27	Test methods - Sampling
prEN 12697-28	Test methods - Preparation of samples for determining binder content, water content and grading
prEN 12697-29	Test methods - Dimensions of a bituminous specimen
prEN 12697-30	Test methods - Specimen preparation by impact compactor
prEN 12697-31	Test methods - Specimen preparation by gyratory compactor
prEN 12697-32	Test methods - Specimen preparation by vibratory compactor
prEN 12697-33	Test methods - Specimen preparation by slab compactor
prEN 12697-34	Test methods - Marshall test
prEN 12697-35	Test methods - Laboratory mixing
prEN 12697-38	Test methods - Common equipment and calibration

Applicable CEN Standards (continued)

	Surface characteristics - Road and Airfield
prEN 13036-1	Test methods - Measurement of pavement macro-texture depth using a patch technique
prEN 13036-2	Test methods - Procedure for determination of skid resistance of a pavement surface
prEN 13036-4	Test methods - Measurement of slip/skid resistance of a surface: The pendulum test
prEN 13036-5	Test methods - Determination of longitudinal evenness parameters or indicators
prEN 13036-6	Test methods - Profilometer-based method for measuring longitudinal evenness
prEN 13036-7	Test methods - Method of measuring surface irregularities: The straight edge test
prEN 13036-8	Test methods - Determining parameters or indicators for transverse evenness: Measurement method

Note: prEN denotes a standard at CEN committee approval stage

Appendix A: Testing aggregates for use in HMA

1 Shape

Flakiness index

A.1 It is desirable that coarse aggregates used in bituminous mixtures have a satisfactory shape and that a large proportion of the material tends to be cubical and not flaky. The Flakiness Index is determined for material passing a 63mm sieve and retained on a 6.3mm sieve (BSI, 1989). The index represents the percentage of the aggregate whose least dimension is less than 0.6 times the mean dimension.

Aggregate angularity

A.2 Two other properties related to the shape of the aggregate are:

- Coarse and Fine Aggregate Angularity; and
- Flat and Elongated Particles.

These shape parameters were considered critical by pavement experts during the study that developed the SHRP Superpave™ Mix Design (Asphalt Institute, 2001).

A.3 A high value of angularity (i.e. more cubical) of both coarse and fine aggregate should produce high levels of internal friction and rutting resistance. Coarse Aggregate Angularity is defined as the percentage by weight of aggregates larger than 4.75mm with one or more fractured faces. Fine Aggregate Angularity is defined as the percentage of air voids in loosely compacted aggregate smaller than 2.36mm. The Superpave™ mix design manual recommends minimum values of angularity with respect to traffic level and the location of the bituminous layer in the road.

Flat and elongated particles

A.4 This characteristic is similar to the flakiness index and is considered important because flat and elongated coarse aggregates are liable to break, either during construction of the pavement or subsequently under traffic. It is defined as the percentage by mass of aggregate (material larger than 4.75mm) that has a maximum to minimum dimension ratio greater than five.

2 Hardness

A.5 Hardness defines the strength or toughness of aggregate particles and can be measured by four tests that are used to establish the ability of an aggregate to resist crushing and impact during road construction and subsequent service life. All four tests are carried out on coarse aggregate particles between 14mm and 10mm only.

Aggregate Crushing Value (ACV)

A.6 In this test (BSI, 1990) a fixed crushing force of 400 kN is applied to the coarse aggregate sample contained within a mould. The ACV test result is reported as the amount of fines produced passing the 2.36mm sieve, expressed as a percentage of the initial sample weight. The test is not suitable for weaker aggregates and should only be used with aggregates that do not produce a compressed lump in the test mould before the maximum specified load has been applied. This test is included in South African specifications (CSRA, 1987) for wearing courses.

10% Fines Aggregate Crushing Test (10%FACT)

A.7 This test (BSI, 1990) and (CSRA, 1986) is a development of the ACV test and uses the same apparatus. Samples are crushed under a range of loads so that the load which produces 10 per cent of fines finer than 2.36mm can be determined. An advantage of the test is that it can be used with all aggregates irrespective of their strength, thus enabling direct comparisons to be made between strong and weak materials.

A.8 An approximate relationship between ACV and 10%FACT is given by the following equation. This relationship is valid in the strength range of 14 to 30 ACV and 100 to 300kN 10%FACT.

$$ACV = 38 - (0.08 \times 10\%FACT)$$

A 10%FACT value of 160kN equates approximately to an ACV of 25 using this relationship.

Aggregate Impact Value (AIV)

A.9 In this test (BSI, 1990) a coarse aggregate sample is subjected to successive blows from a falling hammer to simulate resistance to impact loading. After testing, the AIV is the amount of material finer than 2.36mm expressed as a percentage of the initial sample mass. The test was designed to be supplementary to the ACV test for values up to 26. Softer aggregate should be tested using a modified procedure to ensure that the generation of excessive fines does not invalidate the result.

A.10 The AIV is specified only in the UK where the test is considered to have considerable advantages because the equipment is simple, easily portable and does not require a large crushing press.

Los Angeles Abrasion (LAA)

A.11 In this test (ASTM, C131, C535) an aggregate sample is subjected to attrition and impact by steel balls whilst rotating within a steel cylindrical drum at a prescribed rate for a set number of revolutions. On completion of the test, the sample is screened on a 1.70mm sieve. The coarser fraction is washed, oven dried and weighed. The loss in weight expressed as a percentage of the original sample weight is the Los Angeles Abrasion Value.

3 Durability

A.12 Durability is measured with reference to either mechanical deterioration or a combination of mechanical and physico-chemical deterioration. In the first case it is assessed by abrasion tests in the second by soundness tests.

Aggregate Abrasion Value (AAV)

A.13 This provides an estimate of the surface wear of the aggregate and is particularly relevant for the specification of materials designed to provide good resistance to skidding such as coated chippings, coarse aggregate for porous asphalt and surface dressing chippings. The test method (BSI, 1990) consists of holding a prepared aggregate sample, under a constant load, against a revolving lap with the addition of abrasive sand for a set number of revolutions. The AAV is given by the loss in weight expressed as a percentage of the initial sample.

Polished Stone Value (PSV)

A.14 This is a predictive measure of the susceptibility of aggregate used in wearing courses and surface dressings to polishing under traffic and hence increase the risk of wet skidding at low speeds (BSI, 1989). The recommended value of PSV depends on traffic levels and site characteristics. Accelerated polishing of aggregate samples is achieved by simulating the polishing effect of tyres. A rotating wheel passes over the aggregate sample exerting a total force of 725N. A solution of corn emery and water is fed to the surface of the tyre. The polish of the sample with relation to a control aggregate is measured using a standard pendulum friction tester. Calibration of the friction tester is maintained by the use of the control aggregate which can be obtained from TRL. Maintaining a supply of control and calibration aggregate in developing countries may make the test difficult to sustain.

Water absorption

A.15 High water absorption in aggregates usually indicates low durability and can also cause problems during HMA design. It can be routinely determined as part of the procedure to measure the relative densities of the various size fractions of aggregate (BSI, 1975) since it is the difference in mass between saturated surface dry and oven dried aggregate expressed as a percentage of the oven dried sample mass. In the UK, coarse aggregate having a water absorption of 2 per cent or less is considered durable. A value greater than this necessitates a soundness test to check compliance with specifications. No value of water absorption is given for fine aggregate. South African specifications (CSRA, 1987) distinguish between coarse and fine aggregate, defined as particles larger and smaller than 4.75mm, and call for maximum values of 1 and 1.5 per cent respectively.

Soundness - sodium or magnesium test

A.16 These two tests, which are identical in procedure, can be carried out on both coarse and fine aggregate and they estimate the degree of resistance of the aggregate to in-service weathering. An aggregate sample is exposed to, normally, five cycles of immersion in a saturated solution of either sodium or magnesium sulphate followed by oven drying. The result, calculated from the AASHTO test method (T-104), is the total percentage *loss* of material while the British Standard method (BSI, 1989) reports the percentage material *retained* during the test. The required properties given in Table 4.1 are expressed as percentage material lost during the test.

A.17 Both these tests are considered severe and it is known that they can give variable results depending on aggregate characteristics such as shape, size, porosity and permeability. In reality, the test may measure the number of friable particles among sound aggregates rather than its general performance. Furthermore, the tests are relatively time consuming and expensive. They are normally only applicable where an absolute minimum of aggregate deterioration is required such as on airfields, motorways and trunk roads. However, they may be useful for testing aggregate obtained from new sources and rock which is thought to be susceptible to rapid weathering such as partially degraded basalt.

4 Cleanliness

A.18 Ideally, aggregate should be free of all silt and clay size particles. During HMA production, the ‘free’ silt and clay particles are removed by the dust extraction process or are included as filler. However, any fine material stuck to the aggregate may not be removed and can prevent the bitumen from completely coating the aggregate. Excess clay can also cause ‘balling’ on contact with bitumen.

Decantation test

A.19 This test is a development of the British Standard Sieve Test (BSI, 1985). Initially the dry aggregate sample is agitated to simulate the treatment it receives during transit at the asphalt plant. A deflocculating agent and ultrasonic vibration is then used to dislodge adherent fine particles before wet sieving using a 63µm sieve to determine their proportion.

Sand equivalent value

A.20 This test (AASHTO T176, ASTM D: 2419) is utilised to establish the proportion of detrimental clay-like or plastic fines in fine aggregate passing the 4.75mm sieve. In the test, oven-dried fine aggregate and a solution of calcium chloride, glycerine and formaldehyde are mixed and poured into a graduated cylinder. Agitation loosens the plastic fines from the coarser sand-like particles and, after further addition of solution, the plastic fines are forced into suspension. At the end of a prescribed sedimentation period the heights of sand and clay are measured. The Sand Equivalent Value is the ratio of the height of the sand to clay, expressed as a percentage.

Plasticity index

A.21 This is defined as a range of moisture content, expressed as a percentage of the mass of an oven dried aggregate sample passing a 425µm sieve, within which the material is in a plastic state (BSI, 1990). It is the numerical difference between the liquid and plastic limit of the material. The liquid and plastic limits are difficult to determine for materials of relatively low plasticity and, in such cases, a limit of 2 per cent in the linear shrinkage test will be easier to carry out and to use as a confirmatory test.

5 Bitumen affinity

A.22 Various techniques can be used to test the adhesion between bitumen and aggregate in the presence of water and hence assess the ability of aggregates to resist stripping.

Static Immersion tests

A.23 These tests are not quoted in any aggregate specifications used for HMA. They are generally unreliable both in terms of repeatability and reproducibility and they are more relevant to surface dressing design. If other suitable apparatus is unavailable the AASHTO T182 test may be useful.

A.24 In this Static Immersion test, coarse aggregate is coated with a known amount of bitumen and then immersed in distilled water for 48 hours. At the end of this period the degree of stripping is assessed by visually estimating if the percentage of bitumen left on the chippings is greater or less than 95 per cent. This level was chosen because it is the point where a reasonable degree of reproducibility is achieved.

Immersion strength tests

A.25 In the Immersion Mechanical test (Whiteoak, 1990) Marshall samples are immersed in water, maintained at 60°C, for 48 hours. The stability of the soaked samples is expressed as a percentage of the stability of samples measured by the standard Marshall method, where samples are immersed in water at 60°C for 30 minutes. The samples may first be vacuum treated under water to ensure complete saturation. A minimum value of 75 per cent retained stability should be attained for satisfactory resistance to damage by moisture. A compression test is specified by AASHTO (T165) and ASTM (D1075). There is some doubt as to the usefulness of the tests, particularly with AC designed for heavy traffic conditions where initial voids in the mix (VIM) are at 8 per cent. Tests under vacuum with VIM of 6 per cent may give a better indication of moisture sensitivity.

A.26 A more severe test (AASHTO T283) is specified for Superpave™ mix design (see paragraph D.24) and has been adopted in the Australian provisional mix design procedure. It is considered to give a more reliable indication of moisture sensitivity than the Immersion Mechanical test.

A.27 If an aggregate is known to be susceptible to stripping, or the tests outlined above indicate that this is the case, then 1 to 2 per cent of fresh hydrated lime or Portland cement can be used as part of the filler to improve adhesion.

6 Interpretation of test results

A.28 The recommendations in this Road Note are based on several standards, including British Standard Institution, American AASHTO and ASTM tests, Australian and South African specifications. Tests, even with the same name, may not give comparable results due to subtle differences in the test procedures or materials used in the tests. An example of this is the flakiness index where an immediate complication arises from the definition of the coarse aggregate fraction. The UK test is carried out on material passing the 63mm sieve and retained on a 6.30mm sieve. The South African test calls for material passing 75mm and retained on a 4.75mm sieve.

A.29 It is important, therefore, that authorities ensure that definitions of materials and test specifications are established and are carried out in full accordance with the relevant standard.

Appendix B: Testing bitumens for use in HMA

1 Ageing tests and procedures

B.1 The objective of the laboratory tests described in this Appendix is to ensure that bitumen to be used in HMA will give satisfactory performance in service. Tests are divided into those which are used to specify the required properties of bitumen when it is delivered whilst others specify the limits of acceptable changes in bitumen properties during the various stages of the HMA production process.

Loss on heating test

B.2 This test is part of the UK specifications and is suitable for ranking bitumens according to their tendency to harden and often indicates that a material has been contaminated with light oils. The ageing conditions in the test are akin to those in bulk storage but nothing like those during mixing. In the test, samples of binder are placed on a rotating shelf in a ventilated oven and maintained at 163°C for a period of five hours, whilst the shelf rotates approximately 5-6 times per minute. The samples are approximately 55mm in diameter and 35mm deep. The main disadvantage of the test is that the surface area to volume ratio of the samples is too low and an oxidised skin tends to form quickly on the surface of the bitumen. This skin hampers further oxidation, unlike mixing conditions where homogeneous hardening of the bitumen in thin films occurs on the aggregate.

Thin Film Oven Test (TFOT)

B.3 Practical conditions are simulated somewhat better by this test in that, despite being heated in a similar manner, the bitumen samples are only approximately 3mm thick. It is claimed that the amount of hardening that takes place in this test approximates that obtained in practice during storage and mixing. However, in the test, diffusion into the bitumen film is still limited and it is not possible to obtain homogeneous hardening. The test, therefore, is still far from being ideal.

Rolling Thin Film Oven Test (RTFOT)

B.4 This test was developed by the Californian State Highway Department and simulates the mixing process more closely. In this test cylindrical glass containers holding 35gms of bitumen are fixed on a vertically rotating shelf. During the test the bitumen flows continuously around the inner surface of the container in a relatively thin film with pre-heated air being blown periodically into the container. The normal test procedure uses a temperature of 163°C for a period of 75 minutes. In this manner a homogeneously aged binder is obtained which experience has shown is equivalent to the degree of hardening observed during the mixing and laying of HMA (Dickinson, 1975).

Bitumen durability test

B.5 Developed by the Australian Road Research Board, this test is an extended version of the RTFOT and has been shown to simulate the in-service ageing of the bitumen in thin seals over a period of years. In the test, a small portion of bitumen, already hardened in the RTFOT, is deposited from solvent on the inner walls of the glass container used in the RTFOT to give an even film approximately 20 microns thick. These films are then exposed to the action of air in a RTFOT type oven modified to maintain a temperature of 100°C over long periods. The viscosity of the binder is then tested periodically using a sliding plate viscometer to establish how long it takes until the bitumen reaches a ‘critical viscosity’ (5.7 log Pa.s at 45°C and a shear rate of $5.10^{-3}\ s^{-1}$). The conditioning of the bitumen can take up to 21 days and therefore the test is not specified for the day to day control of binders.

2 Consistency tests

B.6 Bitumens are thermoplastic materials and are characterised by their consistency or ability to flow at different temperatures. The viscosity of a bitumen determines how the material will behave at a given temperature and over a temperature range. The basic unit of viscosity is the Pascal second (Pa.s) where 1 Pa.s = 10 Poise. The absolute (or dynamic) viscosity of bitumen, measured in Pascal seconds, is the shear stress applied to a sample in Pascals divided by the shear rate per second. Viscosity can also be measured in units of m^2/s, or more commonly mm^2/s ($1mm^2/s$ = 1 centistoke). These units relate to kinematic viscosity, usually measured by capillary tube viscometers. Kinematic viscosity is related to absolute viscosity by the expression:

Kinematic viscosity = Absolute viscosity/ Mass density

Penetration test

B.7 This is an empirical test in which a prescribed needle, weighted to 100gms, is allowed to bear on the surface of the bitumen for 5 seconds. The bitumen is held at a temperature of 25°C in a water bath. The depth, in units of 0.1mm, that the needle penetrates is the penetration measurement. As the test temperature rises, the bitumen gets softer and the penetration value is higher. There is a linear relationship between the logarithm of the penetration and temperature defined as (Pfeiffer and Van Doormaal, 1936):

$$\text{Log pen} = AT+C$$

Where, A is the temperature susceptibility, and
C is a constant

B.8 The value of A varies from 0.015 to 0.06, showing that there is a considerable difference between the temperature susceptibility of different bitumens. Pfeiffer and Van Doormaal preferred an expression for the temperature susceptibility which would ensure a value of approximately zero for paving bitumens and, for this reason, defined the Penetration Index (PI) by the equation:

$$50A = \frac{20 - PI}{10 + PI}$$

or

$$PI = \frac{20 - 500A}{1 + 50A}$$

The value of the PI ranges from about -3 for highly temperature susceptible bitumens to about +7 for the least susceptible ones. The value of A, and hence PI, can be derived from penetration measurements at two temperatures, T_1 and T_2, using the equation:

$$A = \frac{\log pen\ at\ T_1 - \log pen\ at\ T_2}{T_1 - T_2}$$

Softening point test

B.9 A number of specifications for penetration grade bitumens also require the softening point of the binder.

B.10 For this test, two samples of bitumen are confined in brass rings, loaded with steel balls, and suspended 25mm above a metal plate in a beaker of water or glycerol. The liquid is then heated at a prescribed rate. As the bitumen softens, the balls and the bitumen gradually sink towards the plate. At the moment the bitumen touches the plate the temperature of the water is determined, and this is designated as the ring and ball softening point. In the ASTM version of the test, the liquid bath is not stirred, as it is in the IP or BS method, and consequently the ASTM results are generally 1.5°C higher than those recorded with the other methods.

B.11 The consistency of bitumen at the softening point temperature ($T_{R\&B}$) has been measured in terms of penetration (Pfeiffer and Van Doormaal, 1936) and the penetration has been found to be 800. Therefore, substituting in the equation above:

$$A = \frac{\log pen\ T_1 - \log 800}{T_1 - T_{R\&B}}$$

Where, $T_{R\&B}$ is the ASTM softening point

The value of pen = 800 at $T_{R\&B}$ is valid for many, but not all bitumens. Bitumens with a high wax content and high PI values, in particular, do not necessarily have a penetration of 800 at $T_{R\&B}$. However, the equation generally provides an alternative, though slightly less accurate way of deriving the PI of a bitumen.

Fraass breaking point test (Fraass, 1937)

B.12 This is one of the very few tests which can be used to describe the behaviour of bitumens at very low temperature. It is essentially a research tool which determines the temperature at which the bitumen reaches a critical stiffness and cracks. In the test, a steel plaque 41 x 20mm, coated with 0.5mm thick film of bitumen, is slowly flexed and released. The temperature of the plaque is decreased at 1°C per minute until the bitumen cracks. The temperature at which the sample cracks is called the breaking point and represents an equi-stiffness temperature. It has been shown that the bitumen has a stiffness of 2.1×10^9 Pa at fracture.

Measurement of bitumen viscosity

B.13 As the relationship between penetration and viscosity is often different for bitumens refined from different crude sources, a number of authorities have adopted bitumen specifications based on viscosity as well as penetration. Viscosity specifications are normally based on a viscosity range measured at 60°C and a minimum value at 135°C. A temperature of 60°C was chosen as it approximates to the maximum temperature of in-service asphalt surfacings and 135°C because it approximates to the temperature at mixing and laydown.

B.14 Two types of viscosity test at 60°C are in common use in the USA and both employ capillary tube viscometers. They are the ***Asphalt Institute Vacuum Viscometer*** and the ***Cannon-Manning Vacuum Viscometer*** and both devices are calibrated using a standard calibrating oil. They work in a similar way by measuring the time taken for the binder, at 60°C, to flow between two timing marks under a prescribed vacuum. This time, when corrected by the calibration factor, gives the value of viscosity in poises.

B.15 Penetration grade bitumens are sufficiently fluid at 135°C to flow through capillary tubes under gravitational forces alone. Therefore a vacuum is not required and a different type of viscometer is used. The one in most common use in the USA is the ***Zeitfuchs Cross-Arm Viscometer***, which is also calibrated with standard calibrating oils. The viscometer is housed in an oil bath maintained at 135°C and the time taken for the bitumen to flow between two points, under gravity, is recorded. The time, corrected by the calibration factor, gives the kinematic viscosity in centistokes. It should be noted that viscosity measurements using this viscometer are expressed in centistokes, whereas those measured at 60°C are in poises. Gravity induces the flow in the kinematic viscometer and therefore the density of the material affects the rate of flow through the capillary tube. The units of poise and stokes or (centipoises and centistokes), are related to each other by the density of the bitumen.

B.16 Viscometers other than capillary viscometers are also in common use. One such instrument is the ***Brookfield Viscometer***. This viscometer is used to specify bitumens in South Africa (SABS, 1997) and is also used in the Superpave design procedure (Asphalt Institute, 2001). The viscosity is determined by measuring the torque required to maintain a constant rotational speed of a cylindrical spindle whilst submerged in bitumen at a constant temperature. The torque is directly related to the binder viscosity, which is read directly from the viscometer. The viscosity can be measured at various test temperatures, however, in the Superpave™ procedure the Brookfield viscometer is used to simply establish that the binder can be handled and pumped at the required temperature, thus measurement at 135°C is specified.

B.17 Another fundamental method of measuring viscosity is the ***sliding plate viscometer***. This apparatus applies a shear stress (Pa) to a film of bitumen sandwiched between two plates and measures the resulting rate of strain (seconds^{-1}). The viscosity in Pascal seconds (Pa.s) is given by shear stress divided by rate of strain. Depending on the load and the size of the sample, viscosities in the range of 10^5 to 10^9 Pa.s can be measured. A special feature of the apparatus is that the shear stress is the same throughout the sample and therefore it can be used to investigate the phenomena of shear stress dependence. Because only small amounts of sample are needed for the test, the sliding plate viscometer has been used extensively for research purposes, however, it is not normally used as a means of specifying penetration grade bitumens for construction purposes.

Ductility

B.18 A number of specifications call for the ductility of the bitumen to be measured. The presence or absence of ductility is usually considered more significant than the actual degree of ductility. Some bitumens having an exceedingly high degree of ductility are also more temperature-susceptible. Ductility of bitumen is measured by an ‘extension’ type of test using a standard size briquette of bitumen moulded under standard conditions and dimensions. It is then brought to a constant temperature, normally 25°C. One part of the briquette is pulled away from the other at a specified rate, normally 5 cm per minute, until the thread of bitumen connecting the two parts of the sample breaks. The elongation (cms) at which the thread breaks is designated the ductility of the bitumen.

The Bitumen Test Data Chart

B.19 Heukelom (1969)(1973) developed a system to enable penetration, softening point, Fraass breaking point and viscosity data to be described as a function of temperature on one chart, known as the Bitumen Test Data Chart (BTDC). The chart consists of a horizontal scale for temperature and two scales for penetration and viscosity. The temperature scale is linear and the penetration scale is logarithmic. The viscosity scale has been devised so that penetration grade bitumens with relatively low PI and low wax contents give straight-line relationships. Figure B1 shows the BTDC with typical temperature-viscosity relationships for three penetration grade bitumens.

B.20 The BTDC shows how the viscosity of a bitumen depends on temperature, but does not account for loading time. Thus, to investigate the effect of temperature only, it is necessary to eliminate the influence of time. This can be achieved with penetration, softening point and the Fraass breaking point tests since the loading times for these are similar. These test data can be combined with viscosity data obtained at temperatures above the softening point because the latter are independent of loading time.

B.21 There are optimum values of bitumen viscosity for the mixing and compaction of dense bituminous mixes. These are illustrated in Figure B2 for a DBM made with 200 pen bitumen. For satisfactory coating of the aggregate the viscosity should be approximately 0.2 Pa.s. During compaction it is widely recognised that the optimal viscosity is between 2 - 20 Pa.s. The BTDC is useful in ensuring that the appropriate operating temperatures are selected to achieve the appropriate viscosity for the bitumen being used.

B.22 The BTDC can also be used to compare the temperature-viscosity characteristics of different types of bitumen. Three classes are usually considered and these are shown in Figure B3. The three classes are Class S (for straight line) bitumens which comprise penetration grade bitumens with limited wax content. Class W (waxy) bitumens, which are generally represented on the BTDC by two lines of equal slope but which are not aligned, and Class B (blown) bitumens which are represented by two intersecting straight lines. Bitumens used for HMA are almost always Class S bitumens.

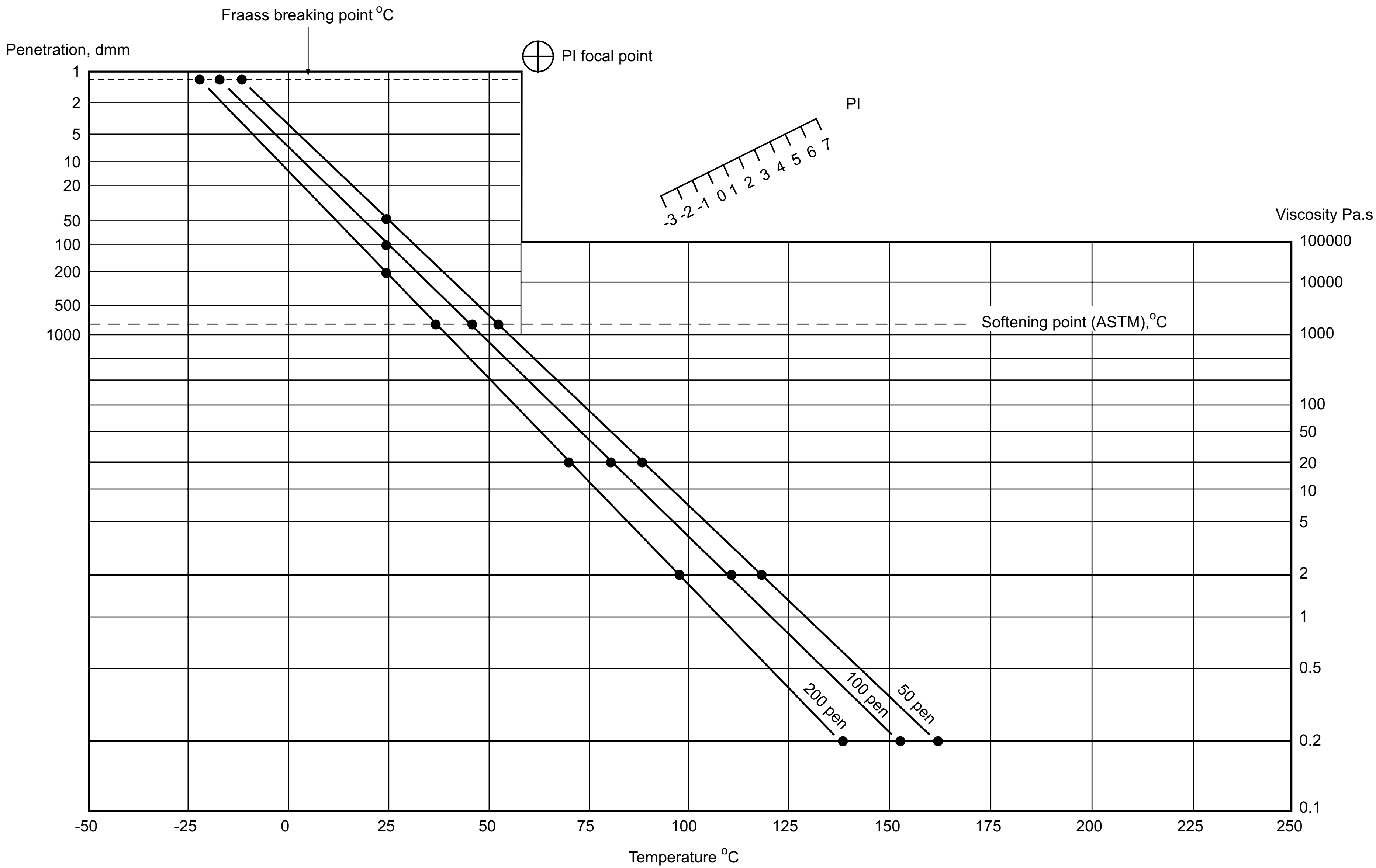

Figure B1 A bitumen test data chart comparing penetration grade bitumens manufactured from one crude (Shell Bitumen Handbook Whiteoak, 1990)

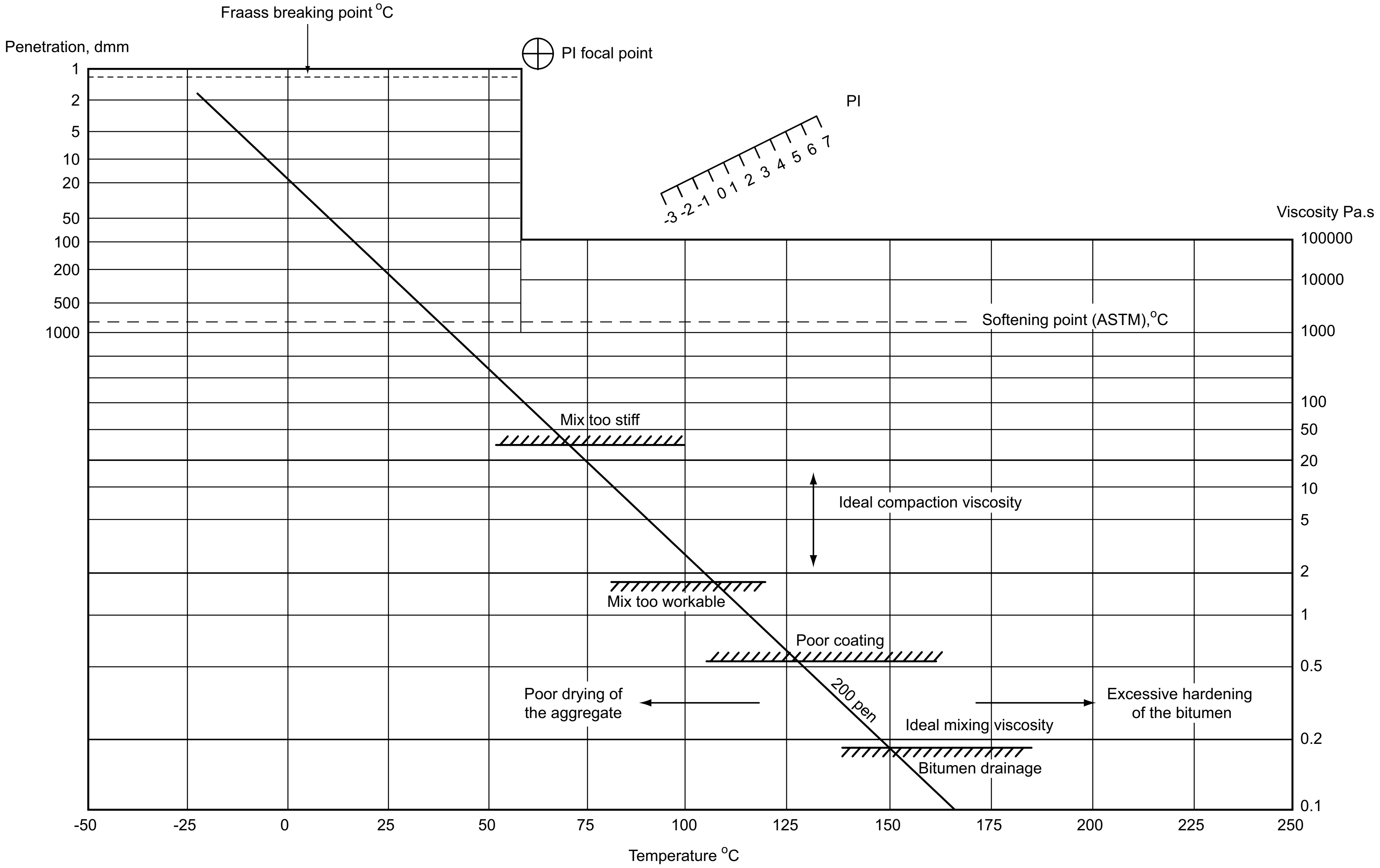

Figure B2 A bitumen test data chart showing 'ideal' bitumen viscosities for optimal mixing and compaction of a dense bitumen macadam (Shell Bitumen Handbook, Whiteoak, 1990)

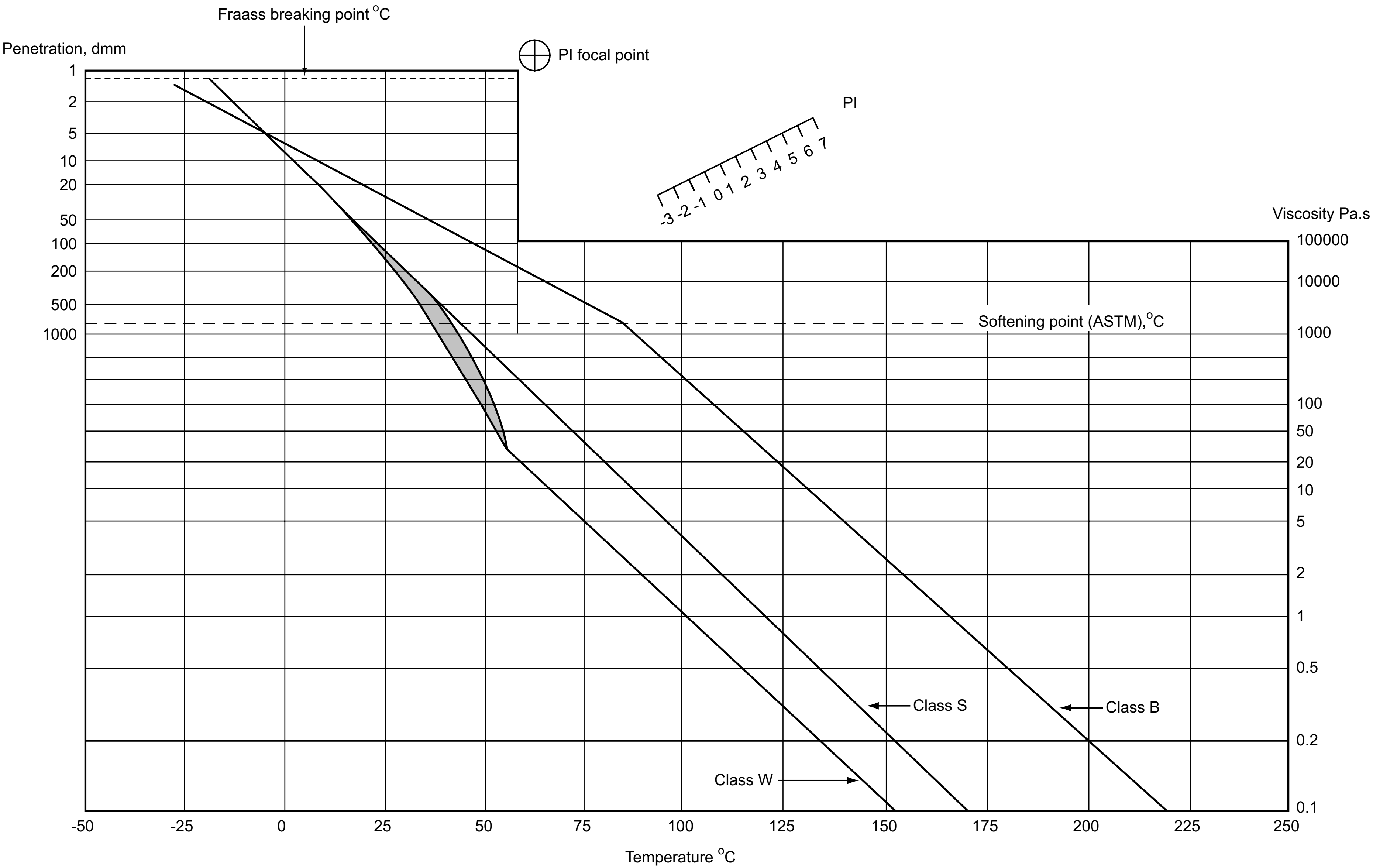

Figure B3 A bitumen test data chart comparing class S, B and W type bitumens (Shell Bitumen Handbook, Whiteoak, 1990)

Superpave™ tests

B.23 The consistency tests described earlier are those most commonly used by the various authorities who have developed test procedures and specifications for penetration grade bitumens. These guidelines, with some modifications, have then been incorporated in local specifications and hence similar relatively low-cost equipment is called for. In comparison, the Superpave™ design procedure (Asphalt Institute, 1997) calls for a more extensive range of viscometer tests that are used to quantify the binder performance at three stages of its life: in its original state, after mixing and construction, and after in-service ageing. The RTFOT is used to simulate the binder ageing that occurs during mixing and construction and the Pressure Ageing Vessel (PAV) procedure is used to simulate the in-service ageing. Table B1 details the test procedures, the purpose of the tests and any pre-conditioning of the binder used in the tests. The viscometer and ageing tests required by the Superpave procedure are relatively expensive and must be used by well trained personnel.

3 Purity tests

B.24 The Solubility Test is a measure of the purity of bitumen. The portion of the bitumen that is soluble in carbon disulphide represents the active cementing constituents. Only inert matter, such as salts, free carbon or non-organic contaminants are insoluble. Due to the hazardous nature of carbon disulphide, trichloroethylene is usually employed in the solubility tests. Determining solubility is simply a process of dissolving 2g of bitumen in 100ml of solvent and filtering the solution through a glass fibre filter. The amount of material retained on the filter is determined by weighing and is expressed as a percentage of the original sample weight.

4 Safety tests

B.25 Normally bitumen is free from water when it leaves the refinery, however, vehicles carrying the bitumen may have moisture in their tanks. If any water is present it will cause the bitumen to foam when heated above 100°C. Bitumen foaming is a safety hazard and a number of specifications require the binder to be free of water and not to foam at 175°C.

B.26 Bitumen, if heated to a high enough temperature, will also release fumes that can ignite in the presence of a spark or open flame. The temperature at which this occurs is called the flashpoint and is normally well above the temperatures used in paving operations. However, to ensure there is an adequate margin of safety, the flash point of the binder is often measured and controlled. The flash point is determined by the ***Cleveland Open Cup*** method in which the sample of bitumen is heated at a constant rate until a test flame, passed across the cup, causes the vapours above the surface to ignite. The lowest temperature at which the test flame causes ignition is taken as the flash point.

5 Precision of test procedures

B.27 The precision to be expected of the recommended tests listed in Table 4.3 are given below;

Penetration test at 25°C - ASTM D 5

i Repeatability

The results of two properly conducted tests by the same operator on the same material of any penetration, using the same equipment, should not differ from each other by more than 4% of their mean, or 1 unit, whichever is the greater.

ii Reproducibility

The results of two properly conducted tests on the same material of any penetration, in two different laboratories, should not differ from each other by more than 11% of their mean, or 4 units, whichever is the greater.

Table B1 Superpave™ binder tests

Equipment	*Binder condition*	*Purpose of test*
Dynamic shear rheometer	Original binder RTFOT aged binder PAV aged binder	Binder properties at high and intermediate temperatures.
Rotational viscometer	Original binder	Binder properties at high temperatures.
Bending beam rheometer	PAV aged binder	Binder properties at low temperatures.
Direct tension tester	PAV aged binder	Binder properties at low temperatures.

Softening Point - ASTM D 36

Using distilled water

i Repeatability

The results of two properly conducted tests by the same operator using the same apparatus on the same sample of bitumen should not differ by more than 1.2°C.

ii Reproducibility

The results of two properly conducted tests from two laboratories on the same sample of bitumen should not differ by more than 2.0°C.

Flash point by Cleveland open cup - ASTM D 92

i Repeatability

The difference between two properly conducted results obtained by the same operator with the same apparatus and the same sample of bitumen should not exceed 8°C.

ii Reproducibility

The difference between two properly conducted tests obtained from different laboratories on the same sample of bitumen should not exceed 17°C.

Solubility - ASTM D 2042

i Repeatability (guide only)

The difference between two properly conducted results obtained by the same operator with the same apparatus and the same sample of bitumen should not exceed 0.1%.

ii Reproducibility (guide only)

The difference between two properly conducted tests obtained from different laboratories on the same sample of bitumen should not exceed 0.26%.

TFOT - ASTM D 1754

Loss by mass

i Repeatability

The results of two properly conducted tests by the same operator on the same sample of bitumen, using the same equipment, should not differ from each other by more than 8% of their mean.

ii Reproducibility

The results of two properly conducted tests on the same sample of bitumen, in two different laboratories, should not differ from each other by more than 40% of their mean.

Percentage of retained penetration

i Repeatability

The results of two properly conducted tests by the same operator on the same sample of bitumen, using the same equipment, should not differ from each other by more than 4%.

ii Reproducibility

The results of two properly conducted tests on the same sample of bitumen, in two different laboratories, should not differ from each other by more than 8%.

Ductility

Values for the repeatability and reproducibility of the ductility test have not been fully developed.

Appendix C: Marshall design method and volumetric design

1 Introduction

C.1 The standard Marshall method is suitable for the design and field control of HMA mixtures containing aggregates with a maximum size of up to 25mm. Aggregates are prepared and blended to give samples which conform to a selected particle size distribution. Initial mix design samples are prepared that cover a range of bitumen contents and are then subjected to a level of compaction which is related to the expected traffic, in terms of equivalent standard axles, to be carried in the design life of the HMA layer(s). The properties of the compacted samples are then determined. These properties include; bulk density, air voids, and stability and deformation characteristics under load. If the mix properties do not meet specified mix design criteria, the mix must be reformulated and the tests repeated until an acceptable design is established.

2 Materials

C.2 For the initial mix design it is advisable to obtain sufficient quantities of coarse aggregate, fine aggregate, filler and bitumen to allow tests to be repeated if necessary or to test different aggregate gradings. For each Marshall design a total of 25kg of aggregate and 5 litres of bitumen are needed to make three briquettes for each bitumen content and to allow for some wastage. The materials must be representative of those to be used on the project.

C.3 For HMA taken from an asphalt plant it is important to complete the Marshall compaction before the samples have cooled below the recommended compaction temperature. Insulated containers of large enough volume should be used for transporting the material to satisfy this requirement.

C.4 It is important to note that the manufacture of test samples with reheated or remoulded materials will not conform to the test procedures upon which this method was developed and may give misleading results.

Aggregates

C.5 Bulk samples taken from each source of nominal size aggregate are reduced in the laboratory by riffling or quartering to give enough material to complete the mix design programme. If additional filler is to be added during production then sufficient material should be obtained from the relevant source for use in the mix design process.

C.6 Representative samples of each aggregate source and filler are subjected to wet sieve analysis and specific gravity tests. It is important that the sieve sizes used for the sieve analysis of the aggregates are the same as those specified in the final mix gradation.

Design of aggregate grading

C.7 Using the results of the sieve analysis obtained for each source of aggregate, a blend is computed which conforms to the specified aggregate particle size distribution. This can be most easily achieved using a computer spreadsheet or by graphical methods such as those described in the Asphalt Institute Manual MS-2. It may be found necessary to change one or more of the aggregate sources to meet the specified particle size distribution.

C.8 The selection of aggregate sources may also be constrained by the number of cold feed bins that are available at the plant. It is preferable to obtain additional cold feed bins rather than pre-mixing two sources of aggregate before placing into a cold feed bin.

Bitumen

C.9 A bulk sample of bitumen should be taken from either the storage tank or the delivery tanker. Bitumen samples should not be kept at the mixing temperature for longer than an hour during any test procedure. It is advisable, therefore, that the bulk sample of bitumen is divided into half-litre containers by pouring at as low a temperature as possible. In this way smaller volumes of bitumen can be heated when required. Containers of cold bitumen should not be heated over naked flames. Heating in an oven or on a sand tray is recommended.

Determination of mixing and compaction temperatures

C.10 The following properties of the bitumen are measured:

i Penetration at 25°C.

ii Softening point (temperature at which penetration is assumed to be 800).

iii Viscosity at approximately 105° to 115°C, 135°C and 160°C.

iv Specific gravity.

The results of tests i) to iii) are plotted on a bitumen test data chart (Whiteoak,1990), illustrated in Appendix B. The plot will indicate the temperature-viscosity characteristics of the bitumen and enable selection of the ranges of ideal mixing and compaction temperatures. The specific gravity of bitumen is required for the volumetric design of the mix.

3 Preparation of test samples

Mass of aggregate required

C.11 The amount of aggregate required for each sample is that which will be sufficient to make compacted specimens 63.5 ± 1.27mm high. This is normally approximately 1.2kg and should be confirmed by compacting a trial sample of 1.2kg of blended aggregate mixed at the estimated optimum bitumen content (see paragraph C.13 below). If the height of the trial specimen falls outside the specified limits then the weight of aggregate used should be adjusted according to the following equation:

$$\textit{Adjusted weight} = \frac{63.5 * \textit{ (weight of aggregate used)}}{\textit{specimen height (mm) obtained}} \qquad \text{C1}$$

C.12 Having determined the weight of aggregate required, a minimum of 21 samples of aggregate complying with the design particle size distribution are placed in metal containers. Fifteen samples are heated to a temperature not exceeding 28°C above the mixing temperature as determined in C10 above.

Design bitumen content

C.13 The design bitumen content for the selected blend of aggregates is determined by testing specimens prepared at bitumen contents which span the expected design value. The expected design value is estimated from the following formula (Asphalt Institute, 1994):

$$DBC = 0.035a + 0.04b + Kc + F \qquad \text{C2}$$

where, DBC = approximate design bitumen content, per cent by total weight of mix

a = per cent of mineral aggregate retained on the 2.36mm sieve

b = per cent of mineral aggregate passing the 2.36mm sieve and retained on the 0.075mm sieve

c = per cent of mineral aggregate passing the 0.075mm sieve

K = 0.15 for 11-15% passing the 0.075mm sieve;

0.18 for 6-10% passing the 0.075mm sieve;

0.20 for 5% or less passing the 0.075mm sieve;

F = 0-2%. Based on absorption of bitumen. In the absence of other data, a value of 0.7 is suggested.

C.14 The aggregate samples are used to make triplicate specimens at the estimated optimum bitumen content and at two increments of 0.5 per cent above and below this optimum. If the estimated bitumen content proves to be different to the actual value then it may be necessary to use the spare aggregate samples to make specimens at one or two additional bitumen contents.

Mixing

C.15 Before mixing, the half-litre containers of bitumen are heated in an oven to the ideal mixing temperature as determined in C10 above. Mixing should be done in a mechanical mixer with a bowl capacity of approximately 4 litres. The mixing bowl, mechanical stirrers and any other implements to be used in the mixing procedure must be pre-heated to the mixing temperature. The heated aggregate sample is placed in the mixing bowl and thoroughly mixed using a trowel or similar tool. A crater is formed in the centre of the mixed aggregate into which the required weight of bitumen is poured. Mixing with the mechanical mixer will then produce a mixture with a uniform distribution of bitumen.

Compaction

C.16 The pre-heated mould, base plate, filling collar and an inserted paper disc should be pre-assembled so that the sample can be compacted immediately after mixing is completed.

C.17 The mould is filled with the mixed material and the contents spaded vigorously with a heated spatula or trowel, 15 times around the perimeter and 10 times over the interior. The surface of the material is then smoothed to a slightly rounded shape onto which another paper disc is placed.

C.18 The temperature of the mix prior to compaction must be within the determined limits (see C10 above). The mould, base plate and filling collar are transferred to the Marshall compaction apparatus and the sample compacted by the specified number of blows of the Marshall hammer. After compaction, the mould assembly is removed and dismantled so that the mould can be inverted. The equipment is reassembled and the same number of blows are applied to the inverted sample. The mould assembly is then placed on a bench where the base plate, filling collar and paper discs are removed.

C.19 The mould and the specimen are allowed to cool in air to a temperature at which there will be no deformation of the specimen during extraction from the mould using an extrusion jack. The compacted briquette is labelled and allowed to cool to room temperature ready for testing the following day. The whole procedure is then repeated on the remaining prepared samples.

4 Testing of specimens

C.20 The briquettes are then tested to determine their volumetric composition and strength characteristics.

Bulk specific gravity determination

C.21 The bulk specific gravity is determined for each briquette at 25°C in accordance with the test procedure described in ASTM D2726.

Stability and flow testing

C.22 After measuring the bulk specific gravity the briquettes are immersed in a water bath at 60°C ± 1°C for 35 ± 5 minutes. Each briquette is then removed in turn and tested on a Marshall crushing apparatus to determine the stability and flow values. The mean value of stability and flow for each triplicate set of briquettes is calculated and recorded.

Determination of VIM

C.23 The maximum specific gravity of the mixes at each bitumen content must be determined to enable VIM to be calculated (see paragraph C.38). After completion of stability and flow tests, two of each triplicate set of briquettes are dried to constant weight in an oven at 105 ± 5°C. Each pair of briquettes is combined to give bulk samples to be tested in accordance with the ASTM D2041 procedure for the determination of maximum specific gravity of the mixes.

Test data

C.24 The test results are plotted and smooth 'best fit' curves drawn. The graphs plotted are:-

i VIM v bitumen content.

ii VFB v bitumen content.

iii VMA v bitumen content.

iv Stability v bitumen content.

v Flow v bitumen content.

vi BSG of mix v bitumen content.

Confirmation of design bitumen content

C.25 The design bitumen content is obtained from the relationship between VIM and bitumen content determined in the Marshall test. The VIM requirement is paramount after which it is necessary to ensure that all of the remaining specified mix criteria are also met (see Chapter 6).

C.26 If any of the criteria are not met or if it is considered that a more economical mix can be designed, then the whole design procedure will have to be repeated using an alternative blend of aggregates, particle size distribution or both.

5 Volumetric analysis

Determination of specific gravity for volumetric analysis.

C.27 Because it is the ***volume*** of the individual components that is important for satisfactory mix design (see Chapter 5), the Bulk Specific Gravity (BSG) of each type of material must be measured so that volumes can be computed from the weights when necessary. The nomenclature and test methods used for volumetric analysis are shown in Table C1.

C.28 Coarse aggregates may have been obtained from more than one quarry and the SG of individual sizes from a common aggregate source may be different. Fine material may be crusher dust, sand or a blend of the two. The mineral filler fraction may be crushed rock or have added material such as hydrated lime or cement, the BSGs of which are very different and must be tested separately.

C.29 Determination of the BSGs of the aggregates is based on the oven dried weight. Accuracy of measurements are important and it is recommended that they are determined to four significant figures, ie three decimal places. If the BSGs of the different aggregate sizes do not differ by more than 0.2 then the inaccuracies produced by proportioning by weight rather than by volume will be small.

C.30 The BSGs of the individual coarse aggregate fractions, the fine aggregate and mineral filler fractions are used to calculate the Bulk Specific Gravity (G_{sb}) of the total aggregate using the following formula:

$$G_{sb} = \frac{P_1 + P_2 + \ldots P_n}{\frac{P_1}{G_1} + \frac{P_2}{G_2} + \ldots \frac{P_n}{G_n}} \qquad \text{C3}$$

where, G_{sb} = bulk specific gravity for the total aggregate.

$P_1, P_2 \ldots P_n$ = individual percentages by weight of aggregates.

$G_1, G_2 \ldots G_n$ = individual bulk specific gravities of aggregates.

C.31 During production of HMA it is essential that the plant produces the same aggregate blend that is adopted for the laboratory design mix. To complete the volumetric analysis of a bituminous mix (see Figure 5.2) it is necessary to determine the maximum specific gravity (G_{mm}) of the loose HMA, the BSG of the

Table C1 Volumetric nomenclature and test methods

Volumetric description	*Nomenclature*	*Determined by test method* *ASTM*	*AASHTO*
a. Constituents			
Bulk Specific Gravity of coarse aggregate	G_{ca}	C127	T85
Bulk Specific Gravity of fine aggregate	G_{fa}	C128	T84
Bulk Specific Gravity of mineral filler	G_f	D854	T100
Bulk Specific Gravity of total aggregate	G_{sb}	–	–
Bulk Specific Gravity of bitumen	G_b	D70	T228
b. Mixed material	G_{mb}		
Bulk Specific Gravity of compacted material			
i Saturated surface dry specimens			T166
ii Wax coated specimens		D2726	
Maximum Specific Gravity of loose material	G_{mm}	D2041	T209
Air voids	VIM	D3203	T269
Effective bitumen content	P_{be}	–	–
Voids in mineral aggregate	VMA	–	–
Voids filled with bitumen	VFB	–	–

compacted material (G_{mb}) and the SG of the bitumen (G_b) used in the mix. Since the laboratory design is based on the volume of the constituents whilst plant operations are based on proportioning by mass, it is important to ensure that any changes to the plant mix comply with volumetric design requirements.

6 Calculation of volumetric properties of mix components

Effective specific gravity of aggregate

C.32 When based on the G_{mm} of a bituminous mixture, the effective SG of the aggregate, G_{se}, includes all void spaces within the aggregate particles, except those that absorb bitumen, and is determined using:

$$G_{se} = \frac{100 - P_b}{\dfrac{100}{G_{mm}} - \dfrac{P_b}{G_b}} \qquad \text{C4}$$

where, G_{se} = effective specific gravity of aggregate.

G_{mm} = maximum specific gravity of mixed material (no air voids).

P_b = bitumen content (percent by total weight of mixture) at which ASTM D2041 test (G_{mm}) was performed.

G_b = specific gravity of bitumen.

Maximum specific gravity of mixtures with different bitumen contents

C.33 The determination of G_{mm} is of paramount importance to volumetric analysis, it is recommended that the determination should be carried out in duplicate or triplicate.

C.34 The G_{mm} for a given mix must be known at each bitumen content to allow the VIM to be calculated. G_{mm} can be measured at each bitumen content and a plot of VMA against bitumen content should produce a smooth relationship. This will indicate if any test result is suspect and that it should be repeated. Asphalt Institute suggest an alternative procedure because the precision of the test is best when the mixture is close to the design bitumen content. By calculating the effective SG (G_{se}) for the measured $G_{mm,}$ using Equation C4 the G_{mm} for any other bitumen content can be obtained as follows:

$$G_{mm} = \frac{100}{\frac{P_s}{G_{se}} + \frac{P_b}{G_b}} \quad \text{C5}$$

where, G_{mm} = maximum specific gravity of mixture (no air voids).

P_s = aggregate content, percent by total weight of mixture.

P_b = bitumen content, percent by total weight of mixture.

G_{se} = effective specific gravity of aggregate.

G_b = specific gravity of bitumen.

Bitumen absorption

C.35 Bitumen absorption is expressed as a percentage by weight of aggregate and is calculated using:

$$P_{ba} = \frac{100(G_{se} - G_{sb})G_b}{G_{se}G_{sb}} \quad \text{C6}$$

where, P_{ba} = absorbed bitumen, percent by weight of aggregate.

G_{se} = effective specific gravity of aggregate.

G_{sb} = bulk specific gravity of total aggregate.

G_b = specific gravity of bitumen.

Effective bitumen content of the mix

C.36 The effective bitumen content does not include absorbed bitumen. It is calculated using:

$$P_{be} = P_b - \frac{P_{ba}P_s}{100} \quad \text{C7}$$

where, P_{be} = effective bitumen content, percent by total weight of mix.

P_b = bitumen content, percent by total weight of mix.

P_{ba} = absorbed bitumen, percent by weight of aggregate.

P_s = aggregate content, percent by total weight of mix.

Percent voids in mineral aggregate (VMA)

C.37 The Voids in Mineral Aggregate includes the volume of air between the coated aggregate particles and the volume of effective bitumen. It is expressed as per cent by weight of total mix using:

$$VMA = 100 - \frac{G_{mb}P_s}{G_{sb}} \quad \text{C8}$$

Where, VMA= voids in mineral aggregate.

G_{mb} = bulk specific gravity of compacted mix.

G_{sb} = bulk specific gravity of total aggregate.

P_s = aggregate content, percent by total weight of mix.

Percent air voids in a compacted mix

C.38 The air voids, VIM, in a compacted mix is the volume of air between the coated aggregate particles. It is calculated using:

$$VIM = 100\left(\frac{G_{mm} - G_{mb}}{G_{mm}}\right) \quad \text{C9}$$

where, VIM = air voids in compacted mix, percent of total volume.

G_{mm} = maximum specific gravity of mix.

G_{mb} = bulk specific gravity of compacted mix.

Percent voids filled with bitumen (VFB) in a compacted mix

C.39 The voids filled with bitumen, VFB, is the percentage of VMA that is filled with bitumen. It is calculated using:

$$VFB = 100\left(\frac{VMA - VIM}{VMA}\right) \quad \text{C10}$$

where, VFB = voids filled with bitumen (per cent of VMA).

VMA= voids in mineral aggregate, per cent of bulk volume.

VIM = air voids in compacted mix, percent of total volume.

7 Worked example for calculating the volumetric components of HMA

Example properties of materials and HMA

C.40 The proportions (P) of the coarse and fine aggregates and filler used in this example together with the individual BSG values are shown in Table C2.

Table C2 Aggregate properties

Aggregate Size	*Percentage by weight of total aggregate*	*Bulk Specific Gravity (oven dried)*
Retained 12.5 mm	5 (P_1)	2.727 (G_1)
Retained 9.5 mm	10 (P_2)	2.731 (G_2)
Retained 4.75 mm	25 (P_3)	2.732 (G_3)
Crusher Dust	48 (P_4)	2.691 (G_4)
Sand	10 (P_5)	2.584 (G_5)
Mineral filler (eg. cement)	2 (P_6)	3.120 (G_6)
Total	100	

C.41 The specific gravity of the bitumen (G_b) used in this example is 1.030.

C.42 The Marshall data obtained from samples using the aggregate proportions shown in Table C2 at five bitumen contents are detailed in Table C3.

C.43 The G_{mm} (ASTM D:2041, AASHTO T209) of the loose material containing 4.5% and 5.0% of bitumen, i.e. the two bitumen contents nearest to the optimum (approximate DBC as defined in Equation C2) were determined as 2.531 and 2.511 respectively.

Calculation of volumetric composition

Bulk Specific Gravity of total aggregate (G_{sb})

C.44 Substituting the data from Table C2 into Equation C3.

$$G_{sb} = \frac{5+10+25+48+10+2}{\frac{5}{2.727}+\frac{10}{2.731}+\frac{25}{2.732}+\frac{48}{2.691}+\frac{10}{2.584}+\frac{2}{3.120}}$$

$$= \frac{100}{36.996} = 2.703$$

C.45 G_{se} is calculated by substituting the values of Gmm at the two test bitumen contents of 4.5% and 5.0% and the specific gravity of bitumen into Equation C4.

Table C3 Marshall properties

% bitumen by total weight of mix (P_b)	*Bulk specific gravity of compacted mix (G_{mb})*	*Stability (kN)*	*Flow (0.25mm)*
3.5	2.386	10.9	8
3.5	2.385	10.7	7
3.5	2.377	11.2	7
Mean	**2.383**	**10.9**	**7**
4.0	2.396	9.7	9
4.0	2.391	10.1	8
4.0	2.408	10.3	8
Mean	**2.398**	**10.0**	**8**
4.5	2.429	10.8	9
4.5	2.389	10.3	9
4.5	2.417	10.4	9
Mean	**2.412**	**10.5**	**9**
5.0	2.427	10.2	9
5.0	2.437	9.7	8
5.0	2.413	10.0	9
Mean	**2.425**	**10.0**	**9**
5.5	2.422	9.8	9
5.5	2.430	10.2	10
5.5	2.435	10.0	9
Mean	**2.429**	**10.0**	**9**

With 4.5% bitumen content:

$$G_{se} = \frac{100-4.5}{\frac{100}{2.531}-\frac{4.5}{1.03}} = \frac{95.5}{35.141} = 2.718$$

With 5.0% bitumen content:

$$G_{se} \frac{100-5.0}{\frac{100}{2.511}-\frac{5.0}{1.03}} = \frac{95}{34.971} = 2.716$$

Mean G_{se} = 2.717

Maximum Specific Gravity (G_{mm}) of mixes with different bitumen contents

C.46 By using Equation C4 and the mean G_{se} calculated above the G_{mm} of the mixes containing 3.5%, 4.0% and 5.5% bitumen can be calculated:

$$G_{mm} = \frac{100}{\frac{96.5}{2.717}+\frac{3.5}{1.03}} = \frac{100}{38.915} = 2.570$$

$$G_{mm} = \frac{100}{\frac{96.0}{2.717}+\frac{4.0}{1.03}} = \frac{100}{39.216} = 2.550$$

$$G_{mm} = \frac{100}{\frac{94.5}{2.717}+\frac{5.5}{1.03}} = \frac{100}{40.121} = 2.493$$

Bitumen absorption (P_{ba})

C.47 Using Equation C6 the bitumen absorption value can be determined:

$$P_{ba} = \frac{100(2.717-2.703)1.03}{(2.717*2.703)} = \frac{100*0.014*1.03}{7.344} = 0.2\%$$

Effective bitumen content (P_{be})

C.48 Using Equation C7 the effective bitumen content can be determined:

For 4.0% bitumen content:

$$P_{be} = 4.0 - \left(\frac{0.2*96}{100}\right) = 3.8\%$$

For volumetric calculations in the Marshall mix design P_{be} is not required.

Voids in Mineral Aggregate (VMA)

C.49 Using Equation C8 the VMA is calculated for each of the five mixes.

With 3.5% bitumen content:

$$VMA = 100 - \left(\frac{2.383}{2.703}\right)*96.5 = 14.9\%$$

To complete the analysis the VMA is calculated for the mixes with bitumen contents of 4.0, 4.5, 5.0 and 5.5%.

Air voids in compacted mix (VIM)

C.50 Using Equation C9 the VIM is calculated for each of the five mixes.

With 3.5% bitumen content:

$$VIM = 100\left(\frac{2.570-2.383}{2.570}\right) = 7.3\%$$

To complete the analysis the VIM is calculated for the mixes with bitumen contents of 4.0, 4.5, 5.0 and 5.5%.

Voids filled with bitumen (VFB)

C.51 Using Equation C10 the VFB is calculated for each of the five mixes.

With 3.5% bitumen content:

$$VFB = 100\left(\frac{14.9-7.3}{14.9}\right) = 51\%$$

To complete the analysis the VFB is calculated for the mixes with bitumen contents of 4.0, 4.5, 5.0 and 5.5%.

Presentation of data

C.52 The complete volumetric and Marshall data are summarised in Table C4.

The test properties in Table C4 are presented graphically in Figure C1.

Table C4 Summary of volumetric and Marshall data

Bitumen content (%)	*Bulk SG of specimen (Gmb)*	*Max SG of loose mix (Gmm)*	*VIM (%)*	*VMA (%)*	*VFB (%)*	*Stability (kN)*	*Flow (0.25mm)*
3.5	2.383	2.570[1]	7.3	14.9	51.0	10.9	7
4.0	2.398	2.550[1]	6.0	14.8	59.5	10.0	8
4.5	2.412	2.531[2]	4.7	14.8	68.2	10.5	9
5.0	2.425	2.511[2]	3.4	14.8	77.0	10.0	9
5.5	2.429	2.493[1]	2.6	15.1	82.8	10.0	9

[1] *Max SG (See paragraph C46)*

[2] *Max SG determined by ASTM or AASHTO method (see paragraph C43)*

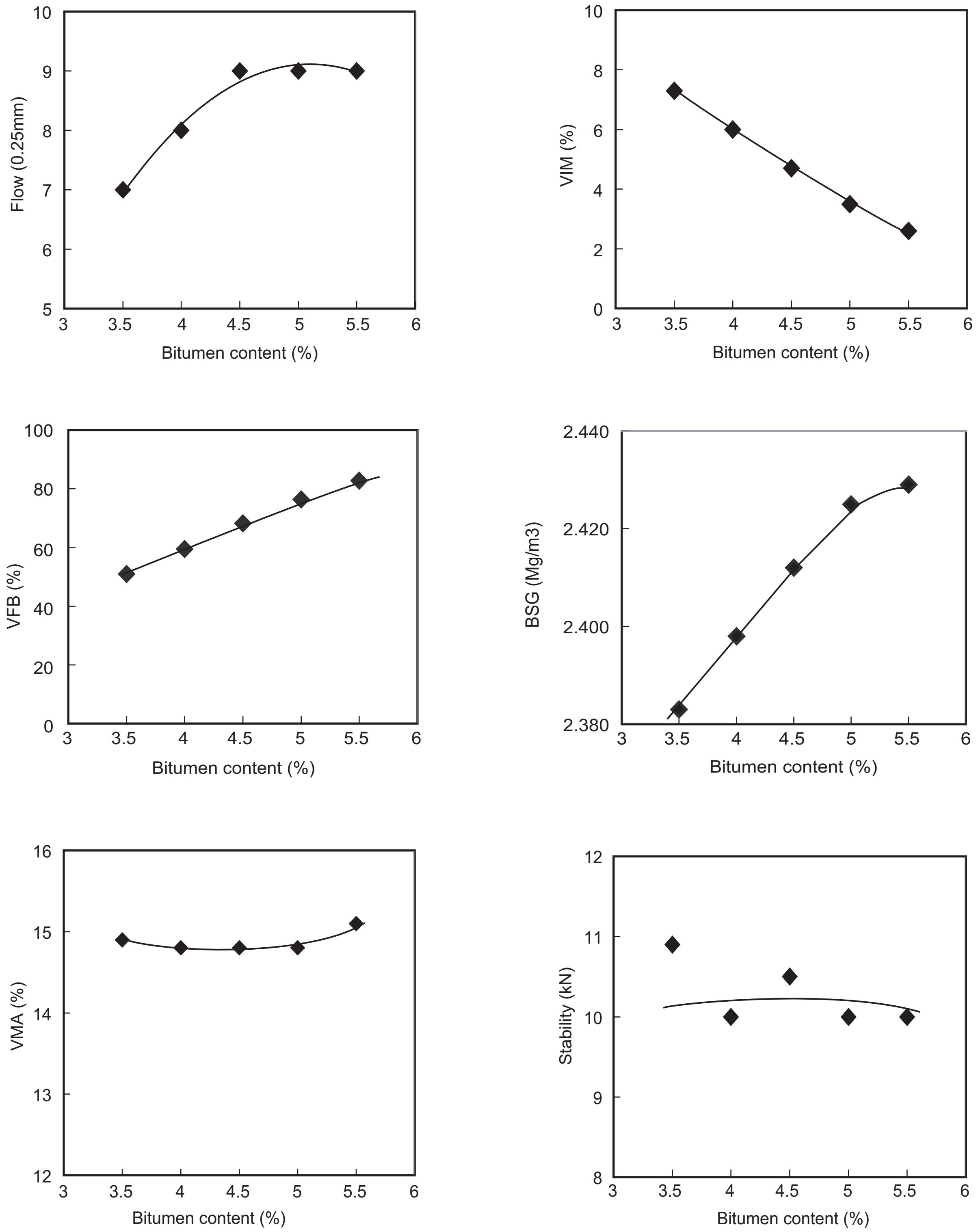

Figure C1 Graphical representation of mix test properties

Trends and relationships of test data

C.53 By examining the test properties graphically (see Figure C1) information can be learned about the sensitivity of the mixture to bitumen content. The trends for each property usually follow reasonably consistent patterns but, in practice, variations can and do occur. In this example:

a The stability value is approximately constant at bitumen contents between 4 and 5.5 per cent. There may be a maximum value at a bitumen content of approximately 4.5 per cent and the high value at 3.5 per cent may be incorrect. This is an example of where the designer might use additional samples to confirm the data.

b The flow value consistently increases with increasing bitumen content. It is advisable to assume that flow will tend to increase at bitumen contents above 5 per cent.

c The curve for the BSG of the total mix increases with increasing bitumen content but would be expected to decrease at higher bitumen contents.

d The percent of air voids, VIM, steadily decreases with increasing bitumen content.

e The percent of voids in the mineral aggregate, VMA, tends to show the expected decrease to a minimum and then increases with increasing bitumen content.

f The percent of voids filled with bitumen, VFB, steadily increases with increasing bitumen content, as the VMA are being filled with bitumen.

Determination of design bitumen content

C.54 The design bitumen content of the mix is selected by considering all of the data discussed previously. As an initial starting point it is recommended that the bitumen content giving 4% air voids is chosen as the design bitumen content (see Chapter 6). All of the calculated and measured mix properties at this bitumen content are determined by interpolation from the graphs shown in Figure C1. The individual properties are then compared to the mix design criteria as specified in Table 6.3.

C.55 Using the data in the worked example, the design bitumen content at 4% VIM is 4.8%. The mix properties at this bitumen content are summarised in Table C5.

Table C5 Mix properties of worked example at 4.8% bitumen content

Mix properties	*Value extrapolated from graphs*
VMA (%)	14.9
VFB (%)	71
BSG (Mg/m^3)	2.419
Stability (kN)	10.1
Flow (0.25mm)	9

Selection of final mix design

C.56 The Asphalt Institute point out that the final selected mix design is usually the most economical one that will satisfy all of the established criteria. However, the mix should not be designed to optimise one particular property but should be a compromise selected to balance all of the mix properties. Table C6 shows the mix properties, the design criteria and the range of bitumen contents over which compliance with the criteria is achieved (obtained from the graphs in Figure C1). This data can be presented in a bar chart such as that shown in Figure C2, which clearly illustrates the effect of variations in bitumen content on the design parameters.

Table C6 Per cent bitumen range complying with mix property criteria

Mix property	*Mix criteria*	*% range of bitumen content giving compliance with MS-2 criteria*
VIM	3.5 - 4.5 %	4.6 – 5.1
VMA	13% minimum	3.5 – 4.9 (remaining on ‘dry’ side[1])
VFB	65 – 75 %	4.3 – 4.9
Stability	8 kN minimum	3.5 – 5.5

1 See Paragraph 6.10

C.57 In this example the VFB criteria are not met if the bitumen content exceeds the design bitumen content by only 0.1 per cent. Normally the production tolerance allowed for variations in bitumen content are ± 0.3 per cent for a wearing course mix and, therefore, this mix may be considered to be too sensitive to errors in bitumen content. There is also doubt about the maximum bitumen content at which VMA will be on the ‘dry’ side of the VMA-bitumen content relationship (see Figure C1).

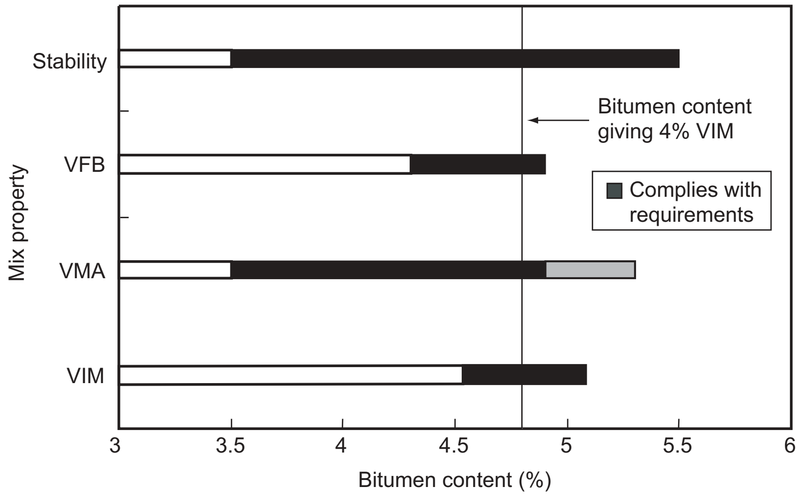

Figure C2 Acceptable bitumen range complying with design criteria

C58 A reduced target bitumen content of 4.6 per cent would be acceptable. This bitumen content would be on the 'dry' side of the VMA-bitumen content relationship, VIM would be 4.2 per cent and VFB would be approximately 70 per cent. The risk of plastic deformation in the resultant mix would be reduced. However, good quality control would be required to ensure that the mix remained within specification.

The preferred solution would be to adjust the aggregate particle size distribution further away from the maximum density line to give slightly more VMA and wider tolerances.

C.59 On any project, pre-construction compaction trials are an essential part of the mix design process and would be used to ensure that the final design mix was satisfactory.

Confirmation of volumetric analysis

C.60 It is important to ensure that the volumetric analysis is correct. As discussed in paragraph 6.5 if there is any doubt about the determined values of VMA then the bitumen film thickness should be calculated to help in the design process. An average bitumen film thickness of 7 to 8 microns can be used as a guide when assessing the suitability of a particular design bitumen content. However, this must be considered together with all evidence from the laboratory design testing and the pre-construction compaction trials before confirming the properties of the target mix.

Bitumen film thickness

C.61 Bitumen film thickness can be estimated using the following formula (CSRA:1987):

$$F = \frac{P_{be}}{100 - P_b} * \frac{1}{A} * \frac{1}{S} * 10^6 \qquad \text{C11}$$

where, F = Film thickness (μm).

P_{be} = Effective bitumen content of HMA (% by mass of mix).

P_b = Total bitumen content of HMA (% by mass of mix).

A = Surface area of aggregate blend (m^2/kg).

S = density of bitumen at 25°C (kg/m^3).

'A', the surface area of the aggregate blend, is calculated from:

$$(2 + 0.02a + 0.04b + 0.08c + 0.14d + 0.3e + 0.6f + 1.6g) * 0.20482 \qquad \text{C12}$$

where, a = percentage passing 4.75mm sieve.

b = percentage passing 2.36mm sieve.

c = percentage passing 1.18mm sieve.

d = percentage passing 0.600mm sieve.

e = percentage passing 0.300mm sieve.

f = percentage passing 0.150mm sieve.

g = percentage passing 0.075mm sieve.

Appendix D: Superpave™

1 Background

D.1 The Strategic Highway Research Programme (SHRP) was a five year, US$150 million project. A third of these funds were directed at studies of bitumen, aggregates, mix design procedures and the equipment necessary for a new mix design methodology called Superpave.

D.2 The Asphalt Institute produced the following manuals for Superpave mix design:

i Performance graded asphalt binder specifications and testing. Superpave Series No.1 (SP-1), 1996.

ii Superpave™ Mix Design. Superpave Series No. 2 (SP-2). Third Edition, 2001.

D.3 Authorities wishing to have detailed knowledge about Superpave mix design must also refer to new AASHTO Standards to ensure that the full requirements of the test methods are obtained. Current standards are listed in Chapter 11. An outline of the procedure is given below to indicate the general methodology.

2 Materials for Superpave™

D.4 All materials are subject to quality and performance based assessments. When applied to bitumens these assessments are related to the results of physical tests. For aggregates the recommendations are largely for 'consensus properties' and source properties, which are outlined in paragraphs D10 and D11.

D.5 Mix acceptance is based on volumetric composition and compaction characteristics which are specified for different levels of twenty-year design traffic loading, expressed in terms of equivalent standard axles (esa).

Selection of grade of bitumen

D.6 The recommended procedure for the selection of the correct grade of bitumen is to determine both high and low pavement design temperatures. The high temperature relates to the pavement temperature at a depth of 20mm below the road surface whilst the low temperature is determined for the surface of the road.

D.7 Methods of determining the road temperatures for design, and levels of reliability, are given in the Manual. The Performance Grade bitumen (or PG binder) is then selected to suit the temperature conditions and this may be further adjusted if traffic loading conditions justify it.

Performance tests for bitumen

D.8 For any given road temperature and traffic loading the selected bitumen must also satisfy specified requirements. These are:

i A minimum flash point temperature.

ii A maximum viscosity of 3Pas at 135°C.

iii Minimum dynamic shear at a temperature appropriate to the road site.

iv After Rolling Thin Film Oven test:

- a maximum percent loss in mass; and
- b minimum dynamic shear at a temperature appropriate to the road site.

v After ageing in a Pressure Ageing Vessel (PAV):

- c maximum dynamic shear at a temperature appropriate to the road site;
- d physical hardening, tests on beams of bitumen;
- e creep stiffness criteria; and
- f direct tension failure criteria.

D.9 Clearly, although the range between high and low design temperatures in tropical countries will often be less than in much of North America, the principles of Superpave can be adopted in developing countries. The equipment required to carry out the bitumen performance tests listed above is relatively complex and expensive and well trained technicians will be needed to operate it. A period of 'calibration' will also be needed. During this time it will be necessary to establish procedures for estimating the appropriate maximum and minimum road surfacing temperatures. Also, it cannot be assumed that there will be a range of bitumens available from which a suitable material can be selected.

Aggregate properties

D.10 Consensus properties were agreed by a selected panel of experts who relied upon their extensive empirical knowledge of factors which are relevant to conditions in the USA. The properties specified, which are also relevant to developing countries with tropical climates are:

i Coarse aggregate angularity.

ii Fine aggregate angularity.

iii Flat/elongated particles.

iv Clay content.

v Combined Bulk Specific Gravity.

vi Combined Apparent Specific Gravity.

Angularity is specified to ensure that good internal friction is obtained in the aggregate structure so as to resist deformation of the asphalt under traffic. Limiting elongation reduces the chances of particle breakage under load and limiting the clay content enhances the bonding between bitumen and aggregate particles.

D.11 Source properties relate to the following properties:

i toughness;

ii soundness; and

iii deleterious material.

Toughness is measured by the Los Angeles abrasion test. Soundness is measured by the sodium or magnesium sulphate soundness test. Deleterious materials are measured by the clay lumps and friable particles test. Aggregate particle size distributions must satisfy the requirements summarised in Tables D1 and D2.

3 Compaction for Superpave™ mix design

D.12 A gyratory compactor is used which provides a method of compaction that is more representative of compaction under road rollers than is the Marshall hammer. The specification of the gyratory compactor is important and the basic requirements for the Superpave compactor are:

i A constant pressure of 600kPa on the compacting ram.

ii A constant rate of rotation of the mould at 30 gyrations per minute.

iii The mould is positioned at a compaction angle of 1.25 degrees.

D.13 In principle, asphalt mixes should be designed to be more resistant to compactive forces as either road temperature or design traffic loading increases.

Table D1 Particle size distributions for Superpave™ HMA (AC type) wearing courses

	Per cent passing sieve size											
	Nominal maximum size (mm)											
	19				*12.5*				*9.5*			
	Control points		*Restricted zone*		*Control points*		*Restricted zone*		*Control points*		*Restricted zone*	
Sieve size (mm)	*Min*	*Max*	*Min*	*Max*	*Min*	*Max*	*Min*	*Max*	*Min*	*Max*	*Min*	*Max*
25	100	–										
19	90	100			100	–						
12.5	–	90			90	100			100	–		
9.5	–	–			–	90			90	100		
4.75	–	–			–	–			–	90		
2.36	23	49	**34.6**	**34.6**	28	58	**39.1**	**39.1**	32	67	**47.2**	**47.2**
1.18	–	–	**22.3**	**28.3**	–	–	**25.6**	**31.6**	–	–	**31.6**	**37.6**
0.600	–	–	**16.7**	**20.7**	–	–	**19.1**	**23.1**	–	–	**23.5**	**27.5**
0.300	–	–	**13.7**	**13.7**	–	–	**15.5**	**15.5**	–	–	**18.7**	**18.7**
0.075	2	8			2	10			2	10		

Table D2 Particle size distributions for Superpave™ HMA (AC type) roadbase and binder courses

Sieve size (mm)	*Roadbase*				*Binder course*			
	Per cent passing sieve size Nominal size (mm)							
	37.5				*25*			
	Control points		*Restricted zone*		*Control points*		*Restricted zone*	
	Min	*Max*	*Min*	*Max*	*Min*	*Max*	*Min*	*Max*
50	100	–						
37.5	90	100			100	–		
25	–	90			90	100		
19	–	–			–	90		
4.75	–	–	**34.7**	**34.7**	–	–	**39.5**	**39.5**
2.36	15	41	**23.3**	**27.3**	19	45	**26.8**	**30.8**
1.18	–	–	**15.5**	**21.5**	–	–	**18.1**	**24.1**
0.600	–	–	**11.7**	**15.7**	–	–	**13.6**	**17.6**
0.300	–	–	**10**	**10**	–	–	**11.4**	**11.4**
0.075	0	6			1	7		

The number of gyrations, defined as 'Initial' ($N_{initial}$), 'Design' (N_{design}) and 'Maximum' ($N_{maximum}$), needed to achieve these three specified levels of compaction should agree with the values shown in Table D3. Other design requirements are also given in Table D4 (AASHTO, MP2-01).

Preparation of mix design samples

D.14 Suitable aggregates and the appropriate grade of bitumen are selected for the traffic loading and temperature regimes at the road location.

D.15 There is no limit to the number of trial aggregate blends that can be tested. In SP-2 it is recommended that three blends are tried and that all of the gradings pass below the restricted zone. The blends are described as coarse, intermediate and fine. The grading of the coarse blend is near the minimum allowable per cent passing the nominal maximum size, the 2.36mm sieve and the 0.075mm sieve. The intermediate grading is not close to any of the control point limits. The fine grading is close to the maximum per cent nominal maximum size and is just below the restricted zone.

Table D3 Superpave™ gyratory compaction effort

Design traffic (esa x 10^6)[1]	*Compaction parameters* $N_{initial}$	N_{design}	$N_{maximum}$
< 0.3	6	50	75
0.3 to < 3[2]	7	75	115
3 to < 30	8	100	160
> 30	9	125	205

1 *Design traffic is 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 esa for 20 years and choose the appropriate N_{design} level.*

2 *The agency may, at its discretion, specify this level of compaction for an estimated design traffic level of between 3 and <10 million esa.*

(See Asphalt Institute, Superpave Manual Series No. 2 (SP-2) for other conditions.

Table D4 Superpave™ HMA design requirements

<table>
<tr><th rowspan="3">Design traffic (esa x 10⁶)[1]</th><th colspan="3">Required relative density (Percent of theoretical maximum specific gravity)</th><th colspan="5">Minimum Voids in Mineral Aggregate (VMA), (percent)</th><th rowspan="3">Voids Filled with Bitumen (VFB)[2] range (Percent)</th><th rowspan="3">Filler to binder ratio range</th></tr>
<tr><th rowspan="2">$N_{initial}$</th><th rowspan="2">N_{design}</th><th rowspan="2">$N_{maximum}$</th><th colspan="5">Nominal maximum aggregate size, (mm)</th></tr>
<tr><th>37.5</th><th>25.0</th><th>19.0</th><th>12.5</th><th>9.5</th></tr>
<tr><td>< 0.3</td><td>≥ 91.5</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>70 – 80[3]</td><td rowspan="5">0.6 – 1.2[5]</td></tr>
<tr><td>0.3 – < 3</td><td>≥ 90.5</td><td rowspan="4">96.0</td><td rowspan="4">≥ 98.0</td><td rowspan="4">11.0</td><td rowspan="4">12.0</td><td rowspan="4">13.0</td><td rowspan="4">14.0</td><td rowspan="4">15.0</td><td>65 – 78</td></tr>
<tr><td>3 – < 10</td><td rowspan="3">≥ 89.0</td><td rowspan="3">65 - 75[4]</td></tr>
<tr><td>10 – < 30</td></tr>
<tr><td>≥ 30</td></tr>
</table>

[1] *Design traffic is the anticipated project traffic level expected on the design lane over a 20-year period. Regardless of the actual design life of the roadway, the design traffic is determined for 20 years.*

[2] *For 37.5mm nominal maximum aggregate size mixtures, the specified lower limit of the VFB shall be 64 percent for all design traffic levels.*

[3] *For 25.0mm nominal maximum aggregate size mixtures, the specified lower limit of the VFB shall be 67 percent for design traffic levels < 0.3 million esa.*

[4] *For 9.5mm nominal maximum aggregate size mixtures, the specified VFB range shall be 73 to 76 percent for design traffic levels $\geq$ 3 million esa.*

[5] *If the aggregate gradation passes beneath the boundaries of the restricted zone specified in Tables D1 or D2, the filler to bitumen ratio range may be increased from 0.6 – 1.2 to 0.8 –1.6 at the agency's discretion.*

D16 It is suggested that, in order to obtain a good understanding of the behaviour of local materials, investigators could also try an aggregate grading that passes through the restricted zone as well as above it.

D17 A method of calculating a trial bitumen content is provided in the AI-SP2 manual. At least two samples of each trial mix are mixed at the appropriate temperature and aged, to represent the effect of plant mixing, by placing them in a forced draft oven for 2 hours ± 5 minutes at a temperature equal to the mixture's compaction temperature ± 3ºC. The mixtures should be stirred after 60 ± 5 minutes to obtain uniform conditioning. Two additional, but uncompacted, samples are made for the determination of maximum theoretical specific gravity.

D.18 The compaction temperature range of an HMA mixture is defined as the range of temperatures where the unaged bitumen has a kinematic viscosity of approximately 0.28 ± 0.03Pa.s measured in accordance with ASTM D4402.

D.19 The samples are compacted to the appropriate number of gyrations selected from Table D3. During compaction the height of the sample is monitored and, knowing the mass of the mix and the volume of the mould, the bulk specific gravity of the mix can be calculated for any number of gyrations.

D.20 After compaction each sample is allowed to partially cool before being extracted from the mould. After fully cooling it's bulk specific gravity (AASHTO T 166/ASTM D 2726) and maximum specific gravity (G_{mm}) AASHTO T 209/ASTM D 2041) are determined.

D.21 Guidance is given on the calculation of volumetric properties for the compacted specimens which will then allow selection of the most suitable aggregate grading.

D.22 A complete mix design, covering a range of bitumen contents, can then be carried out on samples made to the selected grading.

D.23 It is then a simple matter to calculate the volumetric properties of the samples at any number of gyrations and to determine a bitumen content which gives 4 per cent VIM at N_{design}. The criteria which must be met at this bitumen content are summarised in Table D4.

Moisture sensitivity

D.24 The sensitivity to moisture of the design mix is assessed by carrying out the AASHTO T 283 test procedure. Six specimens are compacted to give 7 per cent air voids and three of the specimens are subjected to partial vacuum saturation. For regions which experience cold winters, freezing followed by 24-hour thawing at 60°C is an optional procedure after saturation. The indirect tensile strength of the treated specimens must be at least 80 per cent of that of the remaining three specimens which are not subjected to saturation.

Construction of power grading chart

D.25 A 0.45 power particle size distribution chart can be constructed to suit locally used sieve sizes and Superpave control points and restricted zones can also be included if required.

D.26 Typical sieve sizes referred to in various international and country standards are shown in Table D5 which is appropriate for mixes containing a maximum aggregate size of 50mm. In order to construct a chart for this type of material the following steps should be followed:

i raise the appropriate sieve sizes to the power 0.45 as shown in column 2 of Table D5;

ii scale the converted numbers to the required length of the x-axis, in this example the length is 100 units;

iii plot the chart with a straight line joining the maximum particle size to the origin, i.e. in this example from 100 per cent passing the 50mm sieve to zero per cent and zero sieve size;

iv the x-axis is then marked with the sieve sizes appropriate to columns 1 and 3 in Table D5;

v the Superpave control and restricted zones or nationally specified particle size distribution envelopes can be drawn on the chart if required.

vi If a chart is required for a smaller maximum sized aggregate then the maximum aggregate size is labelled 100 and the smaller sizes are scaled accordingly.

D.27 An example of a 0.45 power chart for a particle size distribution having a maximum aggregate size of 50mm is shown in Figure D1.

Table D5 Data for construction of 0.45 power particle size distributions

			Sieves used in standards				*Superpave (for 50mm max size aggregate)*			
Sieve size (mm)	*Sieve size raised to 0.45 power*	*0.45 power x 100*	*AI*	*BS*	*CEN (UK)*	*CEN (EUR)*	*Control points*		*Restricted zone*	
50	5.81	100	✓	✓				100		
40	5.27	90.45			✓	✓				
37.5	5.11	87.86	✓	✓		✓	90	100		
31.5	4.72	81.23			✓	✓				
28	4.48	77.03		✓						
25	4.26	73.20	✓					90		
20	3.85	66.21		✓	✓					
19	3.76	64.70	✓							
16	3.48	59.88				✓				
14	3.28	56.39		✓	✓					
12.5	3.12	53.59	✓							
10	2.82	48.47		✓	✓	✓				
9.5	2.75	47.36	✓							
8	2.55	43.84				✓				
6.3	2.29	39.37		✓						
6	2.24	38.52			✓	✓				
4.75	2.02	34.67	✓						34.7	34.7
4	1.87	32.09			✓	✓				
3.35	1.72	29.63		✓						
2.36	1.47	25.31	✓	✓			15	41	23.3	27.3
2	1.37	23.49			✓	✓				
1.18	1.08	18.53	✓						15.5	21.5
1	1	17.20			✓	✓				
0.6	0.79	13.67	✓	✓					11.7	15.7
0.5	0.73	12.59			✓	✓				
0.3	0.58	10.00	✓	✓					10	10
0.25	0.54	9.22			✓	✓				
0.212	0.50	8.56		✓						
0.15	0.43	7.32	✓							
0.125	0.39	6.75			✓	✓				
0.075	0.31	5.36	✓	✓			0	6		
0.063	0.29	4.96			✓	✓				

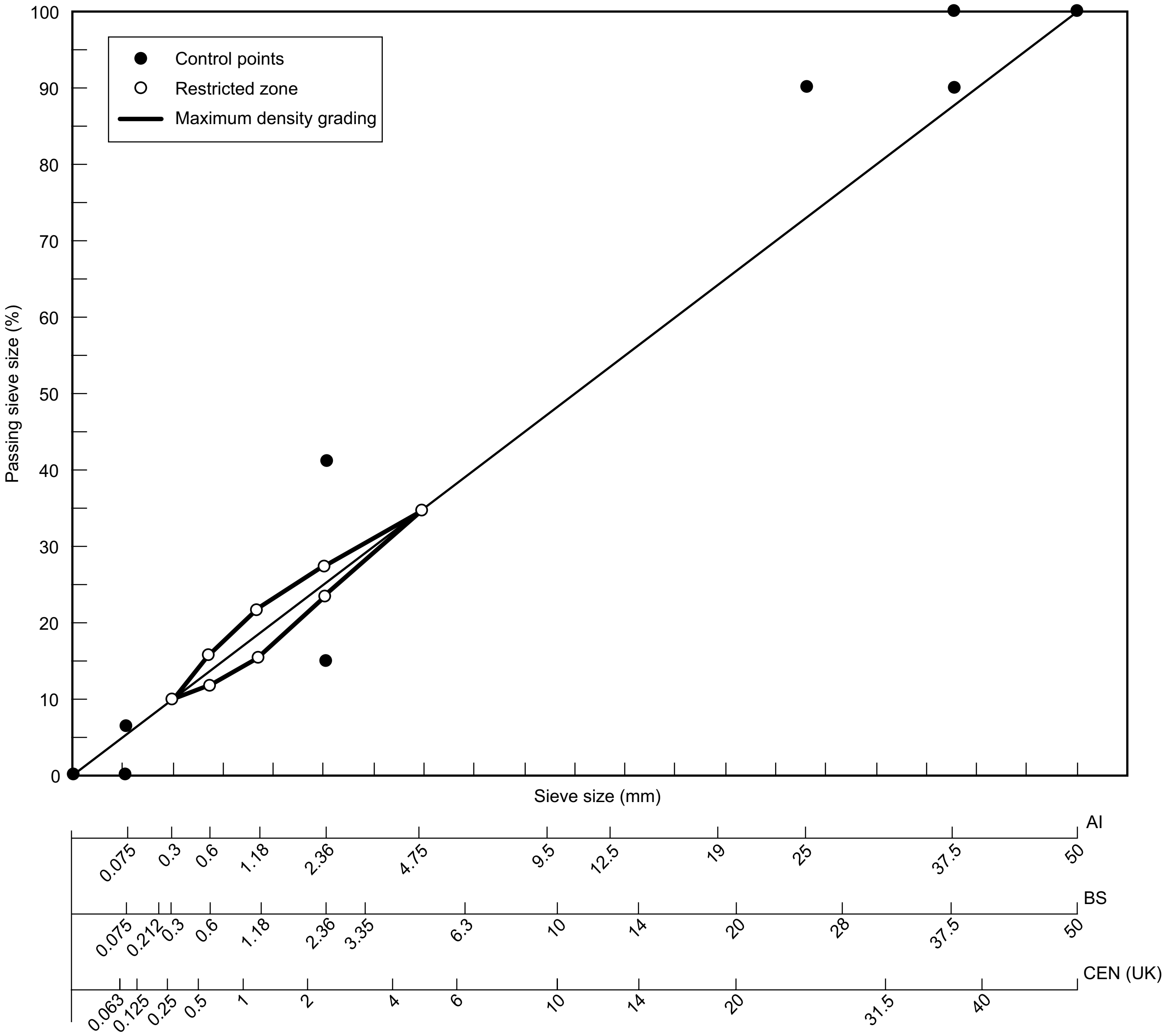

Figure D1 Superpave™ grading limits: 37.5mm Nominal Maximum Size

Appendix E: Performance tests for HMA design

1 Performance tests

E.1 Ensuring that the composition of a mix is correct and that the VIM value will not fall below 3% after trafficking is a vital part of the design process. However, the degree of aggregate interlock and friction between particles also has an important bearing on the resistance of a bituminous mix to shear failure. Although the Marshall design method addresses these problems it has been found that there is poor correlation between measurements of stability and flow and subsequent performance in the road (Whiteoak, 1990). A better indication of the tendency for a mix to deform plastically under traffic is given by the 'Stiffness Quotient' i.e. maximum stability divided by the flow value. However, measurements of both of these values usually have large variations and, therefore, Stiffness Quotient values can also be unreliable.

E.2 Additional performance tests are therefore desirable, particularly for the design of HMA which must carry more than 1 million esa during its design life. Such tests can include:

i determination of mix stiffness moduli;

ii creep;

iii wheel tracking; and

iv shear tests.

E.3 Performance tests do not necessarily guarantee the long term performance of an HMA. The Federal Highway Authority (FHWA), (www.tfhrc.gov. - January 2002) discusses the merits of six design test procedures with particular reference to resistance to rutting. These are:

i Marshall test.

ii Hveem stabilometer.

iii Gyratory Testing Machine.

iv Wheel-track testers.

v Superpave Shear Tester.

vi Creep Tests.

The FHWA points out that none of these tests are capable of predicting rutting of HMA under traffic.

E.4 Design testing is also necessarily carried out on laboratory prepared samples which have not been subjected to traffic loading or to environmental ageing. However, such testing does have considerable potential for 'ranking' mixes and identifying those that would be unsuitable for use.

E.5 Performance tests are being developed by authorities in several countries. It is important to remember that, in many cases different test conditions and requirements are specified even for test methods having a common name and it is, therefore, presently difficult to give generalised specifications for these tests. The following paragraphs describe some of the test methods and the alternative specifications that are used.

Mix stiffness modulus (or resilient modulus)

E.6 Indirect Tensile Tests (ITT) using either the latest version of the British Nottingham Asphalt Tester (BSI, 1993), the Australian Materials Testing Apparatus (MATTA), (Standards Australia, 1994), or other equipment complying with ASTM recommendations offer reliable and appropriate test methods to determine mix stiffness modulus.

Indirect tensile stiffness

E.7 In this test a defined pulse loading is applied vertically across the diameter of the test specimen and the resultant peak transient horizontal diametrical deformation measured. The resilient modulus is normally found to be indirectly proportional to the air void content for a given mix. For laboratory-manufactured specimens, samples with 4 per cent VIM are most relevant to design procedures.

E.8 Different test conditions and performance specifications are recommended by authorities in several countries. The main differences are summarised in Tables E1 to E3.

Specifications for Mix Stiffness (ITT)

E.9 The UK specifications shown in Table E2 below are at present in Draft form and have not been finalised. The specification applies to a macadam material used for the roadbase and binder course layers with a void content of between 7-9%.

E.10 The Australian provisional mix design method is based on the maximum particle size of the aggregate and the grade of bitumen. The grade of bitumen is classified on the basis of the viscosity in Pascal seconds (Pa.s) measured at 60°C. Specifications are under development but typical criteria for mixes containing 4-5% air voids are given in Table E3.

Table E1 Differences between test methods

	Australia	*UK*	*USA*
Reference document	*AS 2891.13.1:1995*	*BS DD 213:1993*	*ASTM D4123:1999*
Minimum No. of specimens required for testing	3	6	3
Core diameter (mm)	100 or 150	100 or 150 or 200	100 or 150
Core depth (mm)	35-90	30-70	50-75
Test temperature (°C)	25	20	5, 25 and 40
Pulse load frequency (Hz)	3	3	0.33, 0.5 and 1
Assumed Poissons ratio	0.35	0.35	0.4

Table E2 UK proposed mix stiffness modulus specification

Bitumen grade	*Criteria*
50 pen	Mean of 6 results must be > 3.5 GPa. No individual result < 2.5 GPa
100 pen	Mean of 6 results must be > 1.1 GPa. No individual result < 0.7 GPa

Table E3 Australian provisional values for mix stiffness modulus

		Maximum particle size (mm)			
Bitumen class	*Viscosity (Pa.s)*	*10*	*14*	*20*	*40*
Cl.170	140-200	2 - 6 Gpa	2.5 - 3.5 GPa	2 - 4.5 GPa	Not applicable
Cl.320	280-360	3 - 6 Gpa	2 - 7 GPa	3 - 7.5 GPa	3.5 - 8 GPa
Cl.600	500-700	3 - 6 Gpa	Not applicable	Not applicable	Not applicable

Creep stiffness modulus

E.11 Creep stiffness modulus is defined as a function of the stress (applied load) and permanent strain (deformation) and can be determined by dynamic tests (static tests are no longer recommended) on core samples. The intended purpose of the test is to provide a means of ranking bituminous mixes in terms of deformation under traffic.

E.12 In the UK a dynamic test procedure, referred to as the RLAT (Repeated Load Axial Test) is currently described in a ‘draft for development document’ (BS DD 226:1996). Dynamic test methods are also referred to in American, Australian and South African standards.

E.13 In the dynamic test a series of pulse load applications are made to the face of the test sample after which the resultant strain is measured. Again there are variations between the test procedures adopted by the different authorities as shown in Table E4.

E.14 The South African National Roads Agency (2001) point out that dynamic creep test results are often extremely sensitive to small variations in measured strains and applied loads. The precision of the loading and measurement devices becomes critical, particularly when values are greater than 20 MPa. It is recommend, therefore, that the dynamic creep modulus is only used as a method of checking mix properties, referring to the data given in Table E5, and not as acceptance criterion. The test is not recommended for mixes containing modified bitumen. It is also recommended that mixes that must have superior resistance to rutting be evaluated using a wheel-tracking test.

E.15 Work in Australia (Oliver *et al*., 1995) showed that the RLAT test was not suitable for assessing the resistance of a mix to deformation because the test could only differentiate between mixes made with different bitumens and the same aggregate grading and not between mixes having different aggregate gradings.

Table E4 Differences between test methods

Test details	*Australia (Provisional)*	*UK*	*US*
Reference document	*AS 2891.12.1-1995*	*BS DD 226:1996*	*ASTM D3497 1999*
Number of specimens required for testing	3	6	3 *
Core diameter (mm)	100 or 150	100, 150 *or* 200	4 x max stone size
Core depth (mm)	50 or 75	<100	2 x core diameter
Test temperature (°C)	50	30	5, 25 and 40
Stress load (kPa)	200	100	Variable up to 240
Pulse load frequency (Hz)	2	2	1 and 4 and 16
Stress applications	Max of 40,000	1800	30-45s duration for each load

** 6 cores are required when taken from a pavement.*

Table E5 Typical values for dynamic creep modulus

Expected rutting resistance	*Dynamic creep modulus (MPa)*
Low	< 10
Medium to Low	10 to 15
Medium to High	15 to 30
High	> 30

Table E6 UK proposed specification for creep stiffness of DBM roadbase and binder course layers

Test conditions	*Maximum strain rate (micro-strain/hr)*	*Maximum strain (%)**
Samples have 7-9% VIM	100	1.5

** [Axial deformation (mm)/height specimen (mm)]x100*

E.16 Work in the UK (Nunn *et al*, 1999) confirmed that RLAT tests did not satisfactorily rank mixes of different composition. However, when a confining force (in this case a vacuum applied to a sample contained within a rubber membrane) was applied the agreement with the wheel-tracking test results was much improved. Test procedures for this constrained creep test (VRLAT) are presently being developed in the UK.

E.17 In addition to variations in test conditions, creep modulus can be calculated for any given number of stress applications and this also contributes to differences in the modulus quoted by each authority. Before authorities adopt a test method it is essential that that they become familiar with the relevant standards and the test equipment used. The climatic conditions and material types found in the country where the test was developed may also be relevant.

Creep modulus specifications

E.18 Because of the effect that VIM has on the creep characteristics of a mix it is necessary that this is specified as part of the test requirements. The UK Draft specification are shown in Table E6.

E.19 The provisional Australian values shown in Table E7 take account of traffic levels as well as pavement temperature and the criteria are based on minimum creep slope values (refer to Australian standard for details of test method). The traffic categories referred to in Table E7 are described in Table E8.

Wheel-tracking test

E.20 The wheel-tracking test has been found to correlate well with the performance of HMA mixes in the field. However, there is no universal criteria that will predict the resistance of HMA to rutting under traffic. Authorities in developing countries need to develop specifications suitable for local conditions and materials.

E.21 A number of wheel-tracking devices are available which apply different loading conditions. It is recommended that wheel-tracking machines should replicate field trafficking as closely as possible and that the tyre on the loading wheel should be of a rubber composition.

Table E7 Australian provisional creep stiffness values

Temperature WMAPT (°C)[1]	*Sample VIM (%)*	*Traffic categories* Very heavy	*Heavy*	*Medium*
>30	4.5 to 5.5	<0.5	0.5 - 3	>3 - 6
20 – 30		<1	1 - 6	>6 - 10
10 – 20		<2	2 - 10	Not applicable

[1] *Weighted mean annual pavement temperature*

Table E8 Australian traffic categories used for creep test interpretation

Traffic category	*Indicative traffic volume (commercial vehicles/lane/day)* Normal	*Stop/start and climbing lanes*
Medium	100 – 500	< 100
Heavy	500 – 1000	100 – 500
Very heavy	> 1000	> 500

E.22 In the British Standard test (BSI 598:Part 110: 1998, which is expected to be superceded by CEN Standard pr12697-22) the sample is subjected to repeated passes of a loaded wheel, specified at 520N, for 45 minutes at a test temperature of 45°C or 60°C. The test is relatively insensitive when used to test AC type mixes at a temperature of 45°C and in many countries which experience high pavement temperatures a test temperature of 60°C will be most appropriate.

E.23 Samples can be 200mm diameter cores or laboratory prepared slabs compacted in a standard purpose-made mould. The samples can, therefore, be obtained in the following ways:

i compacted in a standard rectangular steel mould, with a purpose made quadrant compactor or a pedestrian compaction roller, and tested as a slab or as cores cut from the slab;

ii ‘cores’ compacted in a cylindrical mould in the laboratory; or

iii cores cut from road sites.

During the test the rate, in mm/hour, at which the test wheel penetrates into the sample is recorded and the total depth of penetration is measured at the end of the test. The procedure requires a minimum of six samples to be tested. It is also important that tests carried out to validate mix design are made on material that is representative of the eventual plant mix and, therefore, must be carried out on material taken from the plant.

Specification for wheel-tracking tests

E.24 Current UK specifications are given in Table E9 which provide a useful first approximation.

Adoption of performance tests

E.25 It is clear that authorities in developing countries who are trying to introduce performance testing of HMA must choose test methods with great care. It will be necessary to calibrate the test results for local materials and conditions, using the measured performance of HMA in the field. For such an enterprise to be successful it is essential that laboratory tests and field evaluation are carried out on representative samples and materials. Laboratory and plant mixes must be compared to ensure that the same material is being tested and this means that good quality control of aggregate stockpiles and of material’s sampling must be in place. Test results must be recorded in a suitable manner for future reference and a carefully designed programme of testing should be undertaken to assess the performance of the HMA in the field. Such a programme of research requires considerable resources and takes time. It should, therefore, not be undertaken lightly, but if successful, the benefits can be large.

Table E9 UK wheel-tracking specification

Traffic classification	*Test temperature (°C)*	*Maximum tracking rate (mm/hr)*	*Maximum rut depth (mm)*
Moderate to heavily stressed sites	45	2.0	4.0
Very heavily stressed sites	60	5.0	7.0

Appendix F: Effect of compaction on design bitumen content

1 Marshall compaction

F.1 An important part of the Marshall procedure for the design of AC wearing courses is the selection of the number of blows of the compaction hammer. The level of compaction chosen is meant to replicate the amount of compaction that will occur after several years of trafficking. This is very difficult to allow for when future traffic cannot be reliably estimated. Furthermore current traffic loadings are increasingly exceeding 1×10^6 esa, the lower limit in MS-2 (Asphalt Institute, 1994) which defines heavy traffic.

F.2 Figure F1 indicates the effect of secondary compaction and bitumen content on the final VIM values for a dense wearing course mix.

F.3 When secondary compaction under traffic is underestimated the resulting VIM can be reduced to less than the critical value of 3 per cent and a high risk that plastic deformation will occur. For instance, using 75-blow compaction, the design bitumen content which gives the specified VIM of 4 per cent is 4.15 per cent. However, if secondary compaction were equivalent to 120 or 300 blows in the Marshall test then at the bitumen content of 4.15 per cent, VIM will be reduced to approximately 3.3 per cent and 2.2 per cent respectively. Figure F1 shows that there is a potential for VIM to decrease to 1.5 per cent at a bitumen content of 4.15 per cent when compaction is very high (i.e. equivalent to refusal density in the PRD test).

F.4 It has been recommended in this Road Note that for design traffic in excess of 5×10^6 esa the design VIM should be 5 per cent at 75 blow compaction. Applying this criteria to the data in Figure F1 indicates that the design bitumen content should be approximately 3.75 per cent and VIM should remain above 3 per cent for levels of compaction up to an equivalent of 300 blows of the Marshall hammer.

F.5 It is apparent that the design bitumen contents of 4.15 per cent and 3.7 per cent are quite low indicating that the VMA for this dense wearing course mix is too low. In this situation it would be essential to confirm that determinations of VMA and bitumen film thickness were satisfactory and, from field compaction trials, that the mix is sufficiently workable. An example of the effect of VMA on the relationship between VIM and bitumen content for mixes compacted to refusal density is shown in Appendix G.

2 Gyratory compaction

F.6 It is expected that gyratory compactors will become more commonly available in the future and will eventually replace the Marshall method of compaction. There are important advantages associated with the gyratory compactor, for example:

i two mould sizes with diameters of 100mm and 150mm are available. The larger mould allows the use of aggregate particles larger than 25mm, the limit for Marshall tests; and

ii compaction is more representative of compaction under road rollers and is efficient allowing refusal density tests to be completed in a relatively short period of time.

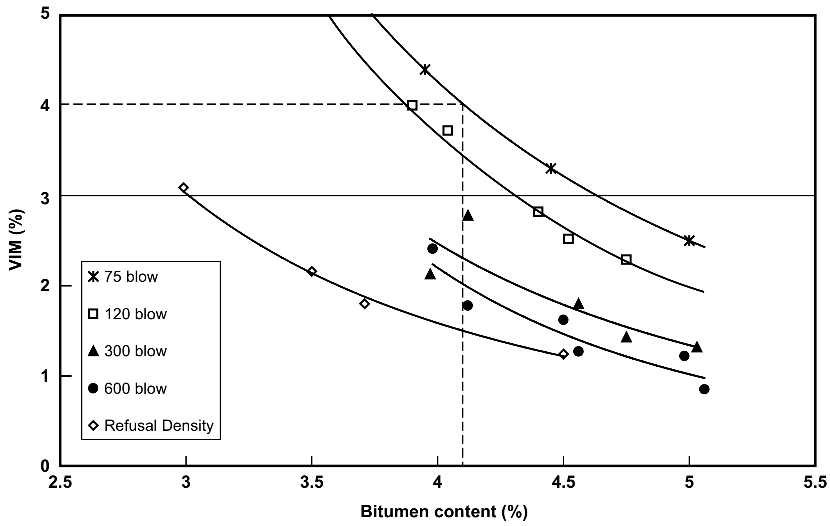

Figure F1 Effect of compaction on mix properties

F.7 Superpave and the provisional AUSTROADS mix design methods are both based on the use of gyratory compactors with particular specifications. It is therefore important for purchasers to ensure that the specifications of a gyratory compactor is compatible with the intended mix design procedures and specifications.

F.8 The principles of mix design do not change when a gyratory compactor is used. Volumetric design of a mix remains of paramount importance. What is needed is knowledge of the numbers of gyrations in the compactor that are required to produce mix densities that are equivalent to those that will be produced after secondary compaction by traffic.

F.9 Authorities will probably have to manage a transitional phase during which both the Marshall method of compaction and the gyratory method will be available. During this phase it is advisable to measure the volumetric properties of mixes compacted by both methods. This can be done by comparing mixes designed by either 50 or 75 blow Marshall compaction to mixes of similar material compacted in a gyratory compactor.

F.10 Research at TRL has shown that the relationship between Marshall compaction and gyratory compaction can vary for mixes with different types of aggregate as can be seen from Figures F2 and F3. The two mixes contained equal percentages of the same fine aggregate but with different coarse aggregates. One mix was made with a fully crushed rock having a good shape and the second contained crushed gravel with smooth micro texture. The result for the mix containing crushed rock aggregate shows that Marshall compaction became increasingly less efficient than gyratory compaction as the bitumen content decreased. In contrast, the mix with gravel aggregate was easily compacted by both methods of compaction over the range of test bitumen contents.

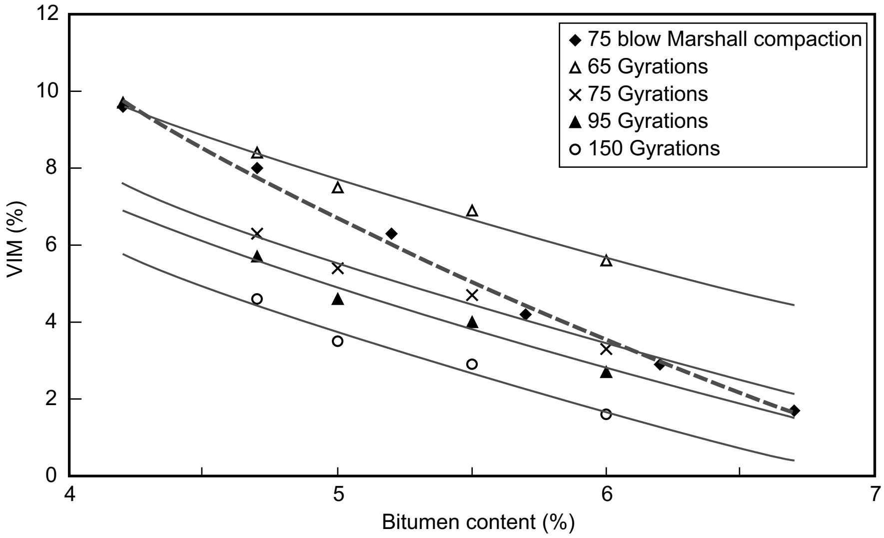

Figure F2 Comparison of Marshall and Gyratory compaction for a mix with cubical aggregate

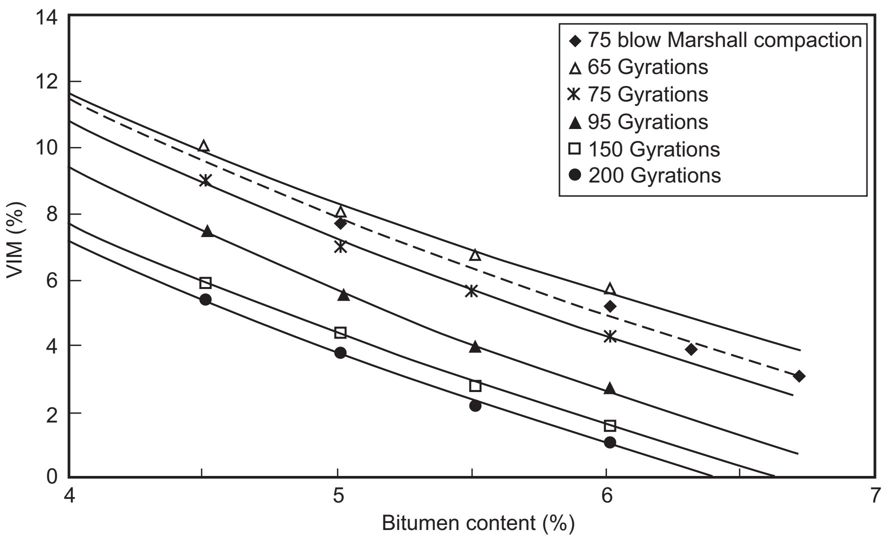

Figure F3 Comparison of Marshall and Gyratory compaction for a mix with gravel aggregate

Appendix G: Refusal density test using a vibrating hammer

1 Equipment

G.1 The equipment and the method of compaction used in the vibrating hammer test procedure for compacting HMA to refusal density is based on the Percentage Refusal Density (PRD) test (BSI, 1989).

A minimum of 8 moulds and 9 base plates are recommended for a refusal mix design. The complete equipment list is:

A tamping foot with a diameter of 102mm.

A tamping foot with a diameter of 146mm.

2 shanks for the tamping feet.

8 No. 152-153mm diameter split moulds.

9 No. base plates.

1 Vibrating hammer with a power consumption of 750 watts or more, operating at a frequency of 50 or 60 Hz.

The equipment can also be used for field control testing.

2 Vibrating hammer compaction

G.2 The refusal density test can be carried out on;

i mixes prepared in the laboratory;

ii hot mix sampled at the asphalt plant or on site; and

iii cores cut from the road.

Compaction of loose mix material

G.3 Trial samples should be made to determine the mass of material required to give a compacted thickness which is approximately the same as the layer to be constructed. As discussed in Chapter 5, the selected maximum stone size in the mix may be influenced by the thickness of the layer to be constructed.

G.4 The moulds, base plate and tamping foot should all be pre-heated and samples should be mixed so that they can be compacted immediately at an initial temperature of 140 ± 5°C for 80/100 penetration grade bitumen and 145 ± 5°C for 60/70 penetration grade bitumen.

G.5 The small tamping foot is used for most of the compaction sequence. The hammer must be held firmly in a vertical position and moved from position to position in the prescribed order, i.e. referring to the points of a compass, the order should be N, S, W, E. NW, SE, SW, NE. At each point compaction should continue for between 2 and 10 seconds, the limiting factor being that material should not be allowed to 'push up' around the compaction foot. The compaction process is continued for a total of 2 minutes ± 5 seconds. The large tamping foot is then used to smooth the surface of the sample.

G.6 To ensure refusal density is achieved the compaction process should then be repeated immediately on the other face of the sample. A spare base-plate, previously heated in the oven, is placed on top of the mould which is then turned over. The sample is driven to the new base plate with the hammer and large tamping foot. The compaction sequence is then repeated. The free base plate should be returned to the oven between compaction cycles.

Compaction of cores to refusal density

G.7 Pre-construction field trials and subsequent monitoring for quality control purposes will involve the compaction to refusal of 150mm diameter cores cut from the compacted surfacing in accordance with the procedure given in BS 598: Part 104:1989. In summary, any material from underlying layers should be removed and the dimensions of the core measured with callipers. The core must then be dried at a temperature which does not cause distortion of the core, but in any event the temperature must not exceed 45°C. Drying for 16 hours at 40°C is normally sufficient to achieve constant mass. This is defined as being a change in mass of no more than 0.05% over a 2 hour period.

G.8 The core is then allowed to cool to ambient temperature and weighed before determining its bulk density. When the core is permeable, which is likely to be the case when samples are taken before trafficking, accurate measurement of bulk density is difficult. BS 598: Part 104 gives the option of coating the core with wax. To make it easier to remove the wax after determining the core's bulk density, it can be cooled in a refrigerator and dusted with talcum powder before waxing. The use of the physical measurements of the core should be considered as an additional or alternative procedure. However, several accurate measurements must be made on each dimension. It is important that an agreed procedure is established at the start of a project.

G.9 After determination of its bulk density and removal of any wax coating, the core is placed in a split mould, heated to the appropriate test temperature, as indicated in paragraph G4, and subjected to refusal compaction as described in G5 and G6. The sample is allowed to cool before removing it from the mould. After reaching ambient temperature, it's bulk density is determined again.

G.10 The Percentage Refusal Density (PRD) is calculated using the following formula:

$$PRD = \left(\frac{Bulk\ density\ of\ core}{Bulk\ density\ after\ PRD\ compaction} \right) \times 100 \qquad \text{G1}$$

G.11 When a core of dense wearing course material is to be compacted to refusal it is probable that it will need to be broken down into a loose state prior to compaction. This is because air voids in dense wearing course mixes often become 'sealed-in' and prevent further densification that may, however, occur under traffic. Initial comparison tests should be carried out on complete and broken down cores to determine if this effect applies to the material being tested.

3 Refusal density design

G.12 The degree of aggregate interlock and friction between particles has an important bearing on the resistance of a bituminous mix to shear failure. For example, uncrushed rounded gravel could meet the minimum VIM requirement when compacted to refusal in a mould, but such a material will have little aggregate interlock and would be expected to suffer shear failure under heavy traffic.

G.13 Firstly, a Marshall design should be carried out to ensure that the aggregate to be used in the production of HMA for severe sites will meet the Marshall design requirements for very heavy traffic given in Chapter 6 of this Road Note.

G.14 Sometimes there is a choice of aggregate sources or sizes for making HMA. In this case the final choice of particle size distribution will be influenced by factors such as workability and sensitivity of the mix to variations in bitumen content. The VMA in a mix has a significant effect on these properties. Figure G1 shows the relationship between VIM, bitumen content and VMA (measured at 3 per cent VIM) for a range of mixes compacted to refusal density. For a design VIM of 3 per cent, mixes with VMA of less than 13 per cent will have a very low bitumen content and will probably be difficult to compact.

G.15 The particle size distributions and the related restricted zones developed in the SHRP programme provide a practical method of describing the characteristics of an aggregate grading. It is important to remember that the SHRP (Superpave™) restricted zone was originally introduced to restrict the amount of rounded pit sand in an asphalt mix. However, it was also recognised that aggregate gradings that avoided the restricted zone would have larger VMA. The choice of particle size distribution will be influenced by the intended layer thickness. It is recommended, therefore, that samples are made to three binder course particle size distributions complying with the requirements of Table D2 and using aggregates from the same sources as those used for the Marshall tests. Two aggregate particle size distributions should pass below the restricted zone by differing degrees and one should pass above the zone. This will provide a range of VMA values and give a good basis for mix selection. If the finer mix meets the criteria it may also prove to be less sensitive to segregation and more durable than the coarser mixes.

G.16 To carry out the mix design it is recommended that duplicate samples are made at the bitumen content which gives approximately 6 per cent VIM in the Marshall test and then at decreasing increments of not more than 0.5 per cent. Tests at four bitumen contents should be sufficient to allow the bitumen content which gives 3 per cent VIM at refusal to be identified. Each sample is subjected to refusal compaction,

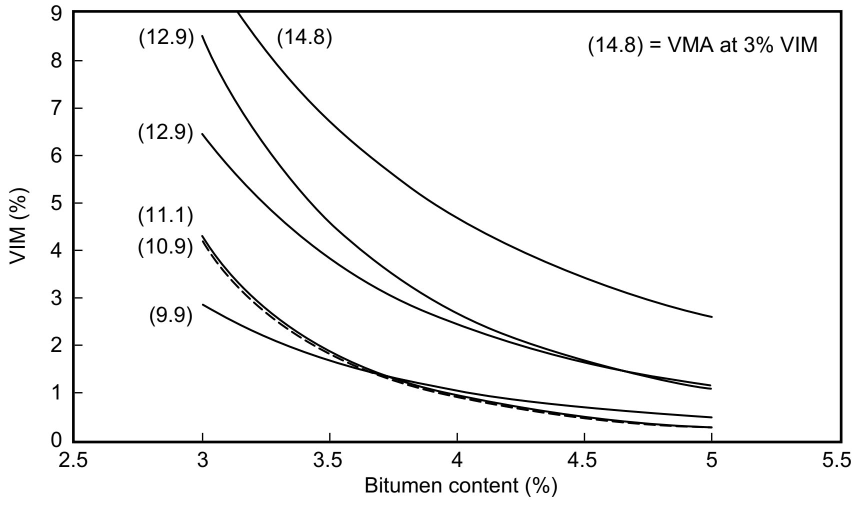

Figure G1 Examples of VIM and VMA relationships for mixes compacted to refusal allowed to cool overnight, and then tested to determine its bulk density. The maximum specific gravity of the mixes (ASTM D:2041) must also be determined (see Appendix C) so that VIM in each compacted sample can be accurately determined.

G.17 The best balance of mix properties will be obtained with the densest mix that can accommodate sufficient bitumen to make the mix workable but which is also as insensitive as possible to variations in proportioning during manufacture and not prone to segregation. Clearly more confidence in mix properties will be gained if the final particle size distribution, allowing for the coarser aggregate, is not dissimilar to the mix used for the Marshall test. If there is any doubt then the Marshall tests can be carried out on the mix, but omitting any material larger than 25mm.

G.18 Whilst designing to refusal density will provide rut resistant mixes, experience may show that designing to 3 per cent VIM at refusal is unnecessarily severe. To improve long-term durability it may be appropriate to design for a higher bitumen content which gives 2 per cent VIM at refusal density. However, accurate determination of VIM is essential and this level of detail will need to be developed based on local experience.

Compaction specifications for HMA designed to refusal density

G.19 The relative level of compaction required in the constructed layer of HMA is based on a comparison of the actual bulk density of a core cut from the compacted layer with the density of the same core after it has been compacted to refusal density. A mix should be laid and compacted on the road to give a mean value of not less than 95 per cent of its refusal density and no individual value should be less than 93 per cent of it's refusal density.

G.20 Because the mix has been designed to refusal density there is every advantage in compacting the mix to the highest density possible. Careful use of vibrating rollers during part of the compaction sequence can make it relatively easy to achieve mean densities above 95 per cent.

Durability of HMA surfacings designed to refusal density

G.21 As described in paragraph G18, the minimum specified density in the compacted layer is 93 per cent of refusal density and, since the target VIM at refusal density is 3 per cent, VIM can be expected to range from 6 to 10 per cent. At the higher values of VIM a mix will be permeable to air and water. The initial rate of compaction under traffic will be an important factor in determining the long term durability of the layer but, because the mix has been designed to be resistant to compaction and because compaction outside of the wheel tracks may be slight, *it is essential* to seal mixes designed by this method as part of the construction process.

G.22 Another factor which will affect long term durability is the degree of age-hardening that will develop during the life of the road. Such hardening will depend upon the VIM at the time of construction, climatic factors and traffic loading at the road site.

G.23 Figure G2 shows the rate of change in bitumen penetration in a DBM layer with a nominal maximum stone size of 37.5mm. The material was laid on a level site, where traffic speeds were high, and was

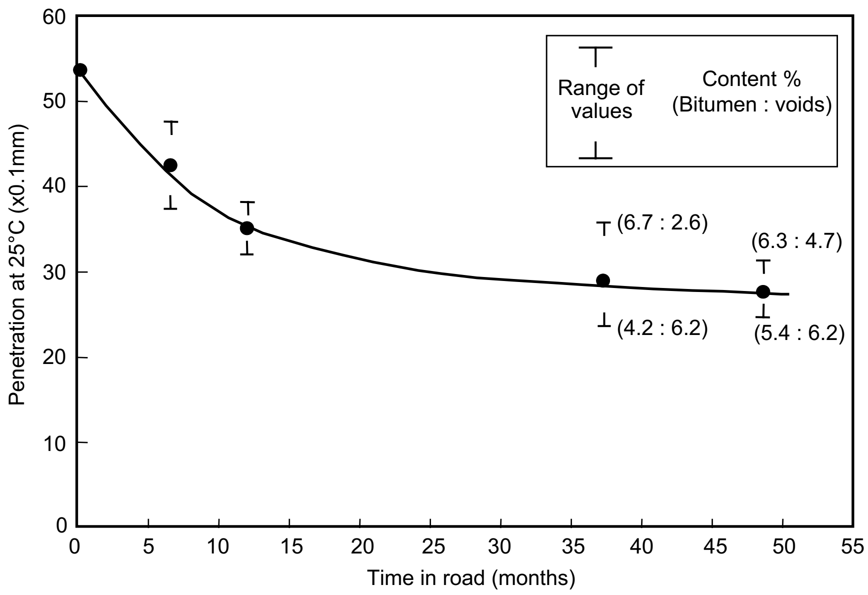

Figure G2 Relationship between age and bitumen penetration for surface dressed bitumen macadam roadbase surfaced with a Cape seal as part of the construction process. The figure shows that even dense mixtures with high bitumen content and low VIM can age harden to penetrations of less than 30 within four years even when sealed. However, without the seal the bitumen in the surface of the DBM layer would become very brittle and almost certainly suffer early 'top down' cracking.

G.24 Where a surface dressing is to be applied it should be constructed as soon as the surfacing is hard enough to prevent excessive embedment of the chippings into the layer. Because HMA designed to refusal density will have a high content of relatively coarse aggregate it should be possible to construct a surface dressing soon after the HMA has been constructed. Surface hardness tests (TRL, 2000 or COLTO, 1998) can be used to determine the optimum time for sealing work. A slurry seal or Cape seal (a slurry seal on a single surface dressing) (TRL, 2000, or COLTO, 1998) can also be used to surface the HMA layer.

4 Transfer of refusal density mix design to compaction trials

G.25 Samples of binder course which have been compacted from the loose state in the laboratory may have densities between 1.5% and 3% lower than for the same material compacted in the road, cored and compacted to refusal in the PRD test. This is an indication of the effect of the different compaction regimes which produce different orientations of the aggregate particles. Refusal densities of laboratory-compacted loose samples and of cores cut from the compaction trials and then subjected to refusal compaction should be compared to determine if this difference occurs. This will ensure that the densities of cores and loose material compacted in the laboratory can be properly compared.

G.26 A minimum of three trial lengths should be constructed with bitumen contents at the laboratory optimum (see paragraph G16 above) for refusal density (giving 3% VIM) and at 0.5% above and below the optimum. The trials should be used to;

i confirm that the mix is workable and can be compacted to a satisfactory density;

ii establish the best rolling patterns for the available road rollers; and

iii obtain duplicate sets of cores so that the maximum binder content which allows 3% VIM to be retained at refusal density can be confirmed.

G.27 For a given level of compaction in the Marshall test, VMA decreases to a minimum and then increases as bitumen content is increased. However, samples compacted to refusal density will usually have relatively constant values of VMA over a range of bitumen contents before the aggregate structure begins to become 'over filled' and VMA increases. This means that during the trials it will be a relatively simple matter to determine the sensitivity of the mix to variations in bitumen content and to confirm the bitumen content required to give a minimum of 3% VIM at refusal density. If necessary the aggregate grading can be adjusted to increase VMA which will reduce the sensitivity of the mix.

G.28 A minimum of 93% and a mean value of 95% of refusal density are recommended as the specification for field compaction of the layer. From these trials and the results of the laboratory tests, it is then possible to establish a job mix formula. After this initial work, subsequent compliance testing based on analysis of mix composition and refusal density should be quick, especially if field compaction is monitored with a nuclear density gauge. This initial procedure is time consuming but is justified by the long term savings that can be made by extending pavement service life and minimising eventual rehabilitation costs.

Appendix H: Recycling of bituminous materials

1 Introduction

H.1 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 can reduce costs and damage to the environment by reducing the exploitation of local natural resources. This is particularly true for countries where there is a shortage of road-building aggregate or where there are no indigenous oil reserves.

H.2 It is possible to ensure that there are benefits both for client and contractor from recycling operations but recycling is unlikely to become widespread in developing countries until certain conditions are met. These include:

i sufficient potential for recycling to justify the purchase of specialist plant;

ii pressure from government agencies to implement recycling;

iii suitable specifications and/or working methodologies for contractual use; and

iv a good understanding of material properties and methods of modification and application.

H.3 In tropical countries 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. Of these types of failure the former is more common, with plastic deformation failures occurring locally where traffic is slow moving such as at junctions and on climbing lanes. 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.

2 Methods of recycling

H.4 Reclaimed Asphalt Pavement (RAP), or millings, are primarily recycled in three ways. They 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.

H.5 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.

H.6 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:

i availability of suitable recycling plant;

ii the thickness of the existing bituminous layer;

iii the effect on traffic management, i.e. can deviations be constructed or must the carriageway be partially open to traffic;

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

v the variability in the properties of the existing material.

H.7 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. The wider tolerances allowed for bituminous roadbase 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 per cent.

H.8 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.

H.9 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.

3 Suggested method of sampling existing asphalt

H.10 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:

i contaminated ‘oil lanes’;

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

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

iv cracking or fretting indicating that appreciable bitumen hardening has occurred.

H.11 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.

H.12 The following tests should be carried out to determine material properties:

i particle size distribution;

ii bitumen content;

iii viscosity of recovered bitumen; and

iv an assessment of crushability.

H.13 Recovering bitumen from RAP to determine the penetration of the existing bitumen will present a problem for many authorities. Unless it is clear that the existing bitumen is severely age hardened it is likely that a carefully selected batch of cores will have to be sent to a qualified testing house to have these tests carried out.

4 Methods of obtaining RAP

H.14 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.

Asphalt millings

H.15 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.

Crushed asphalt

H.16 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.

Granulated asphalt

H.17 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.

5 Stockpiling RAP

H.18 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.

H.19 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. Experience in the USA (NAPA, 1996) has shown that RAP in large piles does not tend to agglomerate. 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.

H.20 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 open-sided building.

H.21 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.

6 Use of RAP as unbound granular material

H.22 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. For example, bitumen in RAP with a penetration value of less than about 15 will behave in a brittle manner.

H.23 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 aggregate in a purpose-made hot-mix recycling plant.

Outline of UK specification for use of RAP as capping layer

H.24 An existing specification (Highways Agency, 1998) and a proposed specification (TRL, 2002) address the use of RAP in capping layers on roads in the UK. A capping layer would only be used in the construction of a new pavement and, in tropical countries, only where the *in situ* subgrade CBR is less than 5 per cent (TRL, 1993).

H.25 The existing UK specification requires milled or granular RAP to meet the grading requirements given in Table H1. The layer can contain 100 per cent RAP provided the bitumen content is less than 10 per cent. The recycled material may be laid to a maximum compacted thickness of 200mm provided the required density is obtained.

H.26 In the UK the required density in the finished layer is obtained by a method specification. This means that the moisture content of the spread material must be within limits determined in standard compaction tests and then the number of passes specified for approved compaction plant must be applied. A minimum density of 95 per cent of the maximum dry density obtained in the British Standard (Heavy) Compaction Test, 4.5 kg rammer, or in the British Standard Vibrating Hammer Test (BS 1377, Part 4, 1990), would be a practical alternative specification.

Table H1 Grading requirements for RAP for use in capping layers

BS sieve size (mm)	*Per cent passing sieve size*
125	100
90	80 - 100
75	65 - 100
37.5	45 - 100
10	15 - 60
5	10 - 45
0.600	0 - 25
0.063	0 - 12

H.27 A new proposed UK specification also requires that RAP meets the grading requirements in Table H1 but other unbound granular material can be added to the RAP to give a material with a reduced effective bitumen content.

H.28 In other countries it is suggested that higher quality materials could be obtained by limiting the maximum effective particle size to 37.5mm by screening out and re-crushing oversized material.

Outline of UK specification for use of RAP as sub-base

H.29 In principle, milled or crushed RAP can be used in the sub-base of a road pavement. The quality of the aggregate in the RAP should meet or exceed normal requirements for these layers. If only good quality aggregate is added to modify the particle size distribution then a compacted layer of the blended material should be of acceptable quality provided that the bitumen in the RAP is hard enough not to hinder compaction and that the finished layer is sufficiently dense.

H.30 In the UK, milled or granulated RAP can be used as sub-base material. The RAP should conform to the particle size distribution given in Table H2.

H.31 Representative samples of the RAP are compacted to determine the optimum moisture content using the procedure described in BS 5835-1: 1980. The laboratory compaction equipment specified in the British Standard includes a special

Table H2 Range of lump-size grading of RAP for use in sub-base

BS sieve size (mm)	*Per cent passing sieve size*
75	100
37.5	85 - 100
20	60 - 90
10	30 - 70
5	15 - 45
0.600	0 - 22
0.075	0 - 10

The lump size distribution shall be determined either by the washing and sieving method or by the dry sieving method of BS 812: Part 103: 1985 (see note 1)

[1] *The plannings should be oven dried (prior to sieving at a temperature of 45 to 50°C. Sieving shall be carried out at a temperature of 20 ± 5°C to reduce the tendency of the bitumen to soften and particles to adhere to each other. The temperature range for sieving can be higher when the RAP is age-hardened.*

mould, a loading frame and a vibrating hammer and would not be readily available in developing countries.

H.32 Where the required laboratory equipment is not available it should be possible to modify the compaction method using the British Standard Vibrating Hammer Test (BS 1377, Part 4, 1990) in conjunction with a mould that will allow drainage during compaction. It would be necessary to prove that this change in methodology is satisfactory and the adoption of the Trafficking Trial procedure could help to achieve this (see paragraph H34).

H.33 Material is then laid at a moisture content between the optimum and 2 per cent below optimum and compacted without drying or segregation.

H.34 The UK specification allows for a Trafficking Trial where this is deemed necessary. RAP, at the correct moisture content, is laid on a prepared trial area constructed to specified standards and trafficked with a loaded truck. After the equivalent of 1000 standard axles have been applied to a single track, the mean deformation in the two wheelpaths is measured. For the material to be approved the mean deformation must be less than 30mm.

Use of RAP as granular roadbase

H.35 If it can be shown that an unbound material containing RAP meets the specifications for grading, density and CBR which are normally applied to fresh materials then it should be acceptable to use the RAP as roadbase. A limiting factor will be the hardness of the bitumen in the RAP; 'softer' bitumen in agglomerations of bitumen and fines may prevent the achievement of the required density. In these circumstances the proportion of RAP used in the new layer will have to be restricted to a level at which thorough compaction can be achieved.

H.36 The roadbase is an important load bearing layer and it is therefore advisable to restrict the general use of RAP in this layer until experience of its performance has been acquired. Inclusion of RAP in a lower roadbase layer or under an AC surfacing on the more lightly trafficked roads would provide an appropriate method of acquiring this experience.

7 Cold mix recycling

H.37 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.

H.38 Cold mix recycling is outside the scope of this Guide and authorities wishing to carry out this type of work should refer to appropriate manuals (see Bibliography) for detailed recommendations.

8 Plant hot mix recycling

H.39 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.

RAP feed to plant

H.40 To avoid blockages that will substantially reduce output, RAP should be metered into the plant through cold feed bins having the following characteristics:

- The sides should be steeper than those of an aggregate feed bin.
- The bottom of the bin may be longer and wider than that of an aggregate feed bin.
- The bottom of the bin may slope downwards, to match an angled feed belt, and the end wall is sometimes left open.
- Vibrators should not be used.
- 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 some time later.

Batch plant recycling

H.41 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:

- The temperature to which the virgin aggregate is heated.

- The temperature and moisture content of the RAP.

- 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 per cent is more typical.

Batch mixers with a separate heating drum (parallel drum)

H.42 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 per cent of RAP to be used in the blend, or even more if a consistent quality of output can be guaranteed.

H.43 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.

9 Evaluation and design - plant hot-mix recycling

Variability of RAP

H.44 As discussed in paragraphs H3 and H10, in tropical countries, RAP will usually either be material which has failed by plastic deformation, and will contain mostly relatively soft bitumen, or badly cracked asphalt containing very hard bitumen. 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.

10 Bitumen rejuvenators

H.45 Rejuvenators have been used to change the properties of bitumen in RAP to those similar to new bitumen. Holmgreen (1980), however, discovered that although such agents could change aged bitumen to the required viscosity, different agents produced binders with different temperature susceptibilities. It was also found that there could be problems relating to the compatibility between aged bitumen and the rejuvenating agent (see also Kallas (1984)).

11 Blending with a soft bitumen

H.46 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 H1 (Whiteoak, 1990):

$$LogP = \frac{A \log Pa + B \log Pb}{100} \qquad \text{H1}$$

where, P = specified penetration of final blend.

Pa = penetration of RAP bitumen.

Pb = penetration of virgin bitumen.

A = percentage of RAP bitumen in the final blend.

B = 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.

Limitations of bitumen blending

H.47 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.

H.48 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.

H.49 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 outlined in Appendix C should be followed. Additional information from a performance test such as the wheel tracking test will also be very helpful in this evaluation.

H.50 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.

12 Mix design

H.51 The most common design procedure used outside Europe is that proposed by the Asphalt Institute (1986). In order to meet mix consistency and design tolerances it is recommended that RAP be used to produce binder course or roadbase mixes for which suitable specifications have been given in Chapter 5 of this Road Note. These recycled materials must be sealed or surfaced with a new bituminous wearing course.

H.52 Initial assessments of the suitability of materials for recycling may necessarily be based on the results of tests carried out on completely ‘broken down’ cores. In practice RAP would be obtained with heavy equipment producing blocks of material for crushing or by use of a milling machine and therefore the actual grading of the RAP must be taken into account when completing the final mix designs. The need for further fine adjustment may be indicated after the handling and compaction characteristics of the new mix have been assessed in pre-construction trials because further breakdown of RAP is likely to have occurred during plant mixing.

H.53 The recycled mix must meet the normal requirements for volumetric composition, i.e. be designed to 4 or 5 per cent VIM and retain at least 3 per cent VIM after secondary compaction by traffic as appropriate (see Appendix C). It will not be possible to test roadbase or binder course mixes by the Marshall method if they contain aggregate particles larger than 25mm. If the proportion of material greater than 25mm is small then the guidance given in paragraphs 6.14 to 6.15 of this Road Note can be adopted and the resultant mix evaluated in field trials.

H.54 Where it is not possible to use the Marshall test because of aggregate size, the Percentage Refusal Test (BSI, 1989) can be used to ensure that a suitable balance between composition and minimum VIM, after compaction, is obtained (see Appendix G). The Percentage Refusal Density Test should be used to check the density of the laid material.

H.55 Aggregate used in the RAP may be known to give good Marshall test results when used in a new AC material. If the fresh aggregate also comes from the same or a similar rock source and meets the normal requirements for aggregate soundness, strength and durability then compositional tests may be sufficient for the design of an asphalt which will perform well under a new asphalt wearing course. However, wherever possible, performance tests such as Indirect Tensile or wheel tracking tests, should be used to ensure that a satisfactory mix can be produced.

13 Recycling feasibility studies

H.56 Two feasibility studies for recycling bituminous surfacing materials are described below. Only core samples could be obtained for testing. Coring locations were established on 1km long sections which were representative of the remainder of the road. Both sections were visually reasonably uniform. A longitudinal and transverse sampling pattern was adopted as shown in Table H3. Site details are given in Table H4.

Table H3 Locations of core sampling

Chainage (m)	*Between v/s w-path and road edge*	*Verge side wheel path*	*Centre-line of lane*	*Off side wheel path*
1000	✓	✓	✓	✓
900		✓		
800	✓	✓	✓	✓
700		✓		
600	✓	✓	✓	✓
500		✓		
400	✓	✓	✓	✓
300		✓		
200	✓	✓	✓	✓
100		✓		
0	✓	✓	✓	✓

Table H4 Details of the road sites

Site	*Traffic category*	*Range of rut depths (mm)*	*Cracking*	*Comments*
Case study 1	Very heavy	0-4	Severe	Failure by cracking of asphalt surfacing.
Case study 2	Very heavy	40-70	None	Climbing lane. Failure by plastic deformation of the asphalt surfacing.

H.57 Structural evaluations should be carried out as part of the feasibility study to ensure that an appropriate method of pavement rehabilitation is selected. However, the absence of deformation on site 1 indicated that the pavement was strong and investigations at site 2 showed that the cement stabilised roadbase had not deformed and that failure was confined to the asphalt layers.

Testing of the core samples

H.58 The cores of RAP were warmed and broken down. The aggregate particle size distributions of the RAP were determined after removal of the bitumen binder from representative samples. Other samples were left in fine 'lump' condition, typical of a fine-milled material, for inclusion in recycled mixes. In practice the effective 'lump size' of RAP depends upon the method of recovery and the degree of breakdown which occurs during mixing.

Case Study 1

H.59 Analyses of core samples are shown in Table H5. As would be expected, the mean bitumen content and the aggregate grading for the wearing and binder course materials were significantly different, but the variability of bitumen content and penetration within each layer was low.

H.60 In practice the wearing and binder courses could be stockpiled separately or as a combined

Table H5 Summary of results: Case Study 1

	Per cent passing sieve size			
	Wearing course		*Binder course*	
BS Sieve (mm)	*Mean*	*Range*	*Mean*	*Range*
28	100	–	97	87-100
20	100	99-100	87	80-96
14	95	92-98	75	66-85
10	87	82-91	63	52-76
6.3	78	70-84	50	40-61
5.0	68	62-77	43	33-53
3.35	55	50-65	38	29-47
2.36	46	42-53	33	26-46
1.18	34	32-37	26	21-32
0.6	27	26-29	22	18-27
0.3	22	20-23	19	16-23
0.212	19	18-21	17	14-21
0.15	18	17-19	15	12-19
0.075	15	14-16	13	10-16
Bitumen (%)	5.4 (sd[1] = 0.2)	4.9-5.7	3.6 (sd = 0.3)	3.1-4.4
Penetration (0.1mm)	13 (sd = 5.1)	6-24	9 (sd = 3.3)	5-15

[1] *sd = standard deviation*

material. Milling would enable separate stockpiling but if simple breaking out equipment were to be used then the two materials would probably be recovered in large lumps which would have to be crushed and mixed to give a single RAP material.

H.61 In this case study the penetration of the bitumen in both layers was less than 15 and the RAP was brittle. It is likely that full-scale recovery of RAP would result in lumps containing material from both layers and the core samples were, therefore, mixed together for testing in the laboratory.

Use as granular material

H.62 Crushing and stockpiling RAP from this site would be relatively easy if the two layers of brittle asphalt were to be combined. An 'all-in' particle size distribution would easily meet the requirements given in Table H2 for sub-base.

H.63 Clearly, thorough pre-crushing of the RAP would make it easier to place and compact the material and vibratory rollers should also be effective in breaking down agglomerations. Fresh aggregate could, if necessary, be blended with the RAP to modify the particle size distribution. The selection of an effective blend of materials must be determined after sufficient field compaction trials have been carried out to ensure that the normal requirements for the density and strength of a sub-base have been achieved.

Hot mix recycling

H.64 During hot mix recycling, agglomerated asphalt remaining in the pre-crushed RAP tends to breakdown and the effective particle size distribution of the RAP will be similar to that used in the laboratory trials described below.

H.65 Reference to historical data for fresh aggregate stockpiles used on a local road contract showed that up to 47 per cent of RAP could be blended with these materials to produce a grading meeting a roadbase specification.

H.66 As indicated in Table H5, the penetration of recovered bitumen ranges from 6 to 24 in the wearing course and from 5 to 15 in the binder course. It is very unlikely that a rejuvenator would suitably modify the bitumen and it would be better to treat the existing bitumen as part of the aggregate particles and to add new 60/70 penetration bitumen.

H.67 A blend of 40 per cent RAP and fresh aggregate was designed to conform with a Superpave™ mix having a nominal maximum aggregate size of 25mm as shown in Table H6. It can be seen that the particle size distribution passes below the restricted zone. The particle size distribution also conforms to the requirements of the Asphalt Institute for a nominal 25mm mix and the resultant mix should, therefore, have the potential to be very stable.

H.68 Mix design was based on the bitumen content which gave 3 per cent VIM at refusal density using a vibrating hammer. The design bitumen content was found to be 2.8 per cent of fresh 60/70 penetration grade bitumen. Samples having a diameter of 150mm were made in a gyratory compactor to give approximately 7 per cent VIM, or 96 per cent of refusal density, for Indirect Tensile Tests (ITT) and wheel tracking tests.

Table H6 An example of blending fresh aggregate and RAP: Case Study 1

	Superpave™ particle size distribution limits			
	Control points		*Restricted zone*	*Blend 40% RAP and fresh aggregate*
	Min	*Max*		
Sieve size (mm)	*Per cent passing sieve size*			
37.5	100	–	–	100
25	90	100	–	97
19	–	90	–	87
4.75			39.5/39.5	33
2.36	19	45	26.8/30.8	23
1.18			18.1/24.1	17
0.6			13.6/17.6	13
0.3			11.4/11.4	10
0.075	1	7	–	7

H.69 The results of the performance tests summarised in Table H7 show that the mix should be very stable under traffic. Wheel tracking tests, in particular, show that the UK specification for ‘Very heavily stressed sites’ (see Appendix E) are easily met.

Case Study 2

H.70 Plastic deformation of up to 70mm had developed at this site. The appearance of the asphalt exposed at the sides of the cores was very uniform throughout the depth of the material and no individual layers could be identified. Asphalt thicknesses are summarised in Table H8.

H.71 Because of the large deformation it is convenient to refer to the position of material in relation to the top of the stabilised roadbase. The cores were sawn into 50mm slices and bitumen content, bitumen penetration and particle size distributions were carried out on the slices. Material in layer 3 represented the top 50mm of the road surfacing. A summary of the results are given in Tables H9 and H10.

H.72 The results show that layer 1 (immediately above the roadbase) had a slightly coarser particle size distribution and a mean bitumen content that was 0.4 and 0.2 per cent lower than for layers 2 and 3

Table H7 Laboratory performance test results for a recycled mix: Case Study 1

No samples	*VIM (%)*	*% of refusal density*	*Wheeltracking rate (mm/hr at 60ºC)*	*ITT (GPa)*	
				At 20ºC	*At 30ºC*
6 per test	6.5-7.3	95.3-96.0	0.17-0.42	5.7-8.0	2.5-3.6

Table H8 Thicknesses of cores: Case Study 2

Chainage (m)	*Core Nos.*	*Core thicknesses (mm)*			
		Near road edge	*Verge-side wheelpath*	*Centre of lane*	*Offside*
0	1		145		
100	2 - 5	190	107	175	115
200	6		106		
300	7 – 10	135	143	145	112
400	11		93		
500	12 - 15	157	113	175	78
600	16		132		
700	17 - 20	148	155	120	150
800	21		95		
900	22 –25	145	105	140	85
1000	26		107		

Table H9 VIM and bitumen content results: Case Study 2

Layer	*Height above roadbase (mm)*	*No. cores analysed*	*No. with VIM <3%*	*Bitumen content (%)*		
				Mean	*SD*	*Range*
1	0 - 50	16	11	4.1	0.3	3.2 - 4.6
2	51 - 100	16	16	4.5	0.4	3.6 - 5.0
3	101 - 150	10	10	4.3	0.4	3.7 - 5.0

Table H10 Aggregate particle size distributions: Case Study 2

	Percent passing sieve size					
	Layer 1		*Layer 2*		*Layer 3*	
BS Sieve (mm)	*Mean*	*Range*	*Mean*	*Range*	*Mean*	*Range*
28	97	91-100	99	96-100	100	–
20	84	70-92	98	86-97	98	94-100
14	70	56-87	80	73-87	89	82-96
10	60	44-74	70	64-75	75	67-85
6.3	51	35-63	60	53-65	60	53-73
5	45	31-58	53	46-60	54	47-65
3.35	40	27-51	47	42-54	48	40-54
2.36	34	23-44	42	37-46	42	35-47
1.18	26	18-34	33	26-46	34	27-38
0.6	22	15-28	28	21-35	29	24-33
0.3	17	12-23	23	16-29	24	18-28
0.212	14	10-19	20	13-25	21	15-25
0.15	11	7-15	16	9-21	18	12-22
0.075	7	5-11	11	5-15	13	7-17

respectively. Large variations were found in the penetration of bitumen recovered from the three layers. Theoretical penetrations of fully blended RAP bitumen and fresh 65 and 100 penetration bitumens calculated using equation H1 are summarised in Table H11.

H.73 Bitumen in layer 1 had a much lower penetration than that in layers 2 and 3 indicating that consistent stockpiles of RAP would be obtained if this layer was treated separately. If 30 per cent of RAP from layer 1 was recycled with fresh aggregate and 100 penetration grade bitumen then substitution of these values in equation H1 indicates that the resultant penetration would be between 53 and 83 with a median value of 66 if full blending of the bitumens occurred.

H.74 A satisfactory mix could probably be made using 30 per cent of well mixed RAP from layers 2 and 3 blended with fresh aggregate and 65 penetration grade bitumen. Although it is very unlikely that uniform blending of the bitumens would be achieved the theoretical penetration of the blended bitumens, before ageing in the mixing process, would range from approximately 49 to 81 with a mean value of 66.

Reclaiming the existing asphalt

H.75 It would be necessary to use a milling machine to produce as fine a material grading as possible because the bitumen in the RAP is soft and simple crushing during hot weather would probably not be possible. Hot mixing would take advantage of the 'soft' bitumen and the addition of fresh 60/70 or 80/

Table H11 Penetration of bitumen in RAP and when fully blendedwith fresh bitumen: Case Study 2

Layer	*Penetration of RAP bitumen*		*Ratio RAP to fresh aggregate*	*Penetration of fresh bitumen*	*Penetration if fully blended after mixing*	
	Median	*Range*			*Range*	*Median*
3. Top 50mm	87	30-135	30:70		52-81	67
			50:50		52-93	69
2. Middle 50mm	68	25-160	30:70	65	49-81	64
			50:50		40-94	65
1. Bottom 50mm	27	12-58	30:70		39-61	49
			50:50		28-59	41
3. Top 50mm	87	30-135	30:70		70-109	90
			50:50		55-115	86
2. Middle 50mm	68	25-160	30:70	100	66-109	87
			50:50		50-116	81
1. Bottom 50mm	27	12-58	30:70		53-83	66
			50:50		35-73	50

Table H12 An example of blending fresh aggregate and RAP: Case Study 2

Sieve size (mm)	*Passing sieve size (%)*	
	Asphalt Institute particle size distribution for 25mm nominal maximum size aggregate	*Blend 50:50 RAP and fresh aggregate*
37.5	100	100
25	90 - 100	93
12.5	56 - 80	66
4.75	29 - 59	34
2.36	19 - 45	22
0.300	5 - 17	12
0.075	1 - 7	6

Table H13 Laboratory performance test results for a recycled mix: Case Study 2

No samples	*VIM (%)*	*% of refusal density*	*Wheeltracking rate (mm/hr at 60ºC)*	*ITT (GPa) At 20ºC*	*At 30ºC*
6 per test	≏ 7	96.0	0.12-0.46	–	1-2

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A guide to the design of hot mix asphalt in tropical and sub-tropical countries

OS-E