2013

PAVEMENT DESIGN MANUAL VOLUME I: FLEXIBLE PAVEMENTS

THE FEDERAL DEMOCRATIC REPUBLIC OF

ETHIOPIA

ETHIOPIAN ROADS AUTHORITY

PAVEMENT DESIGN MANUAL

VOLUME I
FLEXIBLE PAVEMENTS

2013

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FOREWORD

The road network in Ethiopia provides the dominant mode of freight and passenger transport and thus plays a vital role in the economy of the country. The network comprises a huge national asset that requires adherence to appropriate standards for design, construction and maintenance in order to provide a high level of service. As the length of the road network is increasing, appropriate choice of methods to preserve this investment becomes increasingly important.

In 2002, the Ethiopian Roads Authority (ERA) first brought out road design manuals to provide a standardized approach for the design, construction and maintenance of roads in the country. Due to technological development and change, these manuals require periodic updating. This current version of the manual has particular reference to the prevailing conditions in Ethiopia and reflects the experience gained through activities within the road sector during the last 10 years. Completion of the review and updating of the manuals was undertaken in close consultation with the federal and regional roads authorities and the stakeholders in the road sector including contracting and consulting industry.

Most importantly, in supporting the preparation of the documents, a series of thematic peer review panels were established that comprised local experts from the public and private sector who provided guidance and review for the project team.

This Manual supersedes the Pavement Design Manual Volume I Flexible Pavements and Gravel Roads part of the ERA Design Manuals of 2002. The standards set out shall be adhered to unless otherwise directed by the concerned bodies within ERA. However, I should emphasize that careful consideration to sound engineering practice shall be observed in the use of the manual, and under no circumstances shall the manual waive professional judgment in applied engineering.

On behalf of the Ethiopian Roads Authority I would like to take this opportunity to thank DFID, Crown Agents and the AFCAP team for their cooperation, contribution and support in the development of the manual and supporting documents for Ethiopia. I would also like to extend my gratitude and appreciation to all of the industry stakeholders and participants who contributed their time, knowledge and effort during the development of the documents. Special thanks are extended to the members of the various Peer Review Panels whose active support and involvement guided the authors of the manual and the process.

It is my sincere hope that this manual will provide all users with both a standard reference and a ready source of good practice for the pavement design of roads, and will assist in a cost effective operation, and environmentally sustainable development of our road network. I look forward to the practices contained in this manual being quickly adopted into our operations, thereby making a sustainable contribution to the improved infrastructure of our country.

Comments and suggestions on all aspects from any concerned body, group or individual as feedback during its implementation is expected and will be highly appreciated.

Zaid Wolde Gebriel

Director General, Ethiopian Road Authority

PREFACE

The Ethiopian Roads Authority is the custodian of the series of technical manuals, standard specifications and bidding documents that are written for the practicing engineer in Ethiopia. The series describe current and recommended practice and set out the national standards for roads and bridges. They are based on national experience and international practice and are approved by the Director General of the Ethiopian Roads Authority.

This *Pavement Design Manual Volume I Flexible Pavements - 2013* forms part of the ERA series of Road and Bridge Design documents. The complete series of documents, covering all roads and bridges in Ethiopia, includes the following documents:

1. Route Selection Manual
2. Site Investigation Manual
3. Geotechnical Design Manual
4. Geometric Design Manual
5. Pavement Design Manual Volume I Flexible Pavements
6. Pavement Design Manual Volume II Rigid Pavements
7. Pavement Rehabilitation and Asphalt Overlay Design Manual
8. Drainage Design Manual
9. Bridge Design Manual
10. Low Volume Roads Design Manual
11. Standard Environmental Procedures Manual
12. Standard Technical Specifications
13. Standard Drawings
14. Best Practice Manual for Thin Bituminous Surfacings
15. Standard Bidding Documents for Road Work Contracts – A series of Bidding Documents covering the full range of projects from large scale works unlimited in value to minor works with an upper threshold of $300,000. The higher level documents have both Local Competitive Bidding and International Competitive Bidding versions.

These documents are available to registered users through the ERA website: www.era.gov.et

Approach to Manual Updates

The following principles have guided the preparation of this revision to the Flexible Pavement Design Manual:

- The manual provides design details for pavement structures that are known to be successful in the overall environment in which the manual is to be used. The manual provides choices but each choice is not likely to be suitable in all situations. For example, the manual does not cater for situations where specifications are not enforced on site (although it does provide guidance on which pavement structures are easier to build and provides information on how quality control can be improved).

- Additional Specifications are required for implementation of the manual recommendations on site. These Specifications are normally a subset of the details in the manual, but with extra criteria to cater for construction-related issues.
- The manual includes pavement structures that might not be used in the near future in Ethiopia but may be required in the longer term, for example when traffic levels increase. The inclusion of these structures in the current version is intended to encourage demonstration projects and further research.
- Local short term bad experience of the use of one type of structure has not been used as a reason to exclude it from the manual if wider international experience shows that it has been successful in similar environments. Pavement structures that are currently expensive in Ethiopia are also not specifically excluded: they might become more competitive in the future.

The pavement design charts have been reorganised into four groups for the current version of the manual. The groups are based on the type of surfacing and the structure of the pavement. Many of the layer thicknesses in the charts are the same as in the charts from the 2002 edition, but all thicknesses have been re-checked. Minor edits have been made for consistency as traffic and subgrade strength changes from cell to cell. In view of the revisions to the charts, Appendix H has been completely rewritten.

Manual Updates

Significant changes to criteria, procedures or any other relevant issues related to new policies or revised laws of the land or that is mandated by the relevant Federal Government Ministry or Agency should be incorporated into the manual from their date of effectiveness.

Other minor changes that will not significantly affect the whole nature of the manual may be accumulated and made periodically. When changes are made and approved, new page(s) incorporating the revision, together with the revision date, will be issued and inserted into the relevant chapter.

All suggestions to improve the manual should be made in accordance with the following procedures:

1. Users of the manual must register on the ERA website: www.era.gov.et
2. Proposed changes should be outlined on the Manual Change Form and forwarded with a covering letter of its need and purpose to the Director General of the Ethiopian Roads Authority.
3. After completion of the draft review period, proposed modifications will be assessed by the requisite authorities in ERA.
4. Agreed changes will be approved by the Director General of the Ethiopian Roads Authority on recommendation from the Deputy Director General (Engineering Operations).
5. All changes to the manual will be made prior to release of a new version of the manual.
6. The release date will be notified to all registered users and authorities.

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CHANGE CONTROL DESIGN MANUAL

MANUAL CHANGE			This area to be completed by the ERA Director of Quality Assurance
Manual Title:______________________________ ____			CHANGE NO.__________________ SECTION NO.__________________
Section Table Figure Page	**Explanation**	**Suggested Modification**	

Submitted by: Name:__________________________________Designation:______________________________

Company/Organisation ____

Address ____

_______________________________email:_____________________________________Date:________

Manual Change Action

Authority	**Date**	**Signature**	**Recommended Action**	**Approval**
Registration				
Director Quality Assurance				
Deputy Director General Eng. Ops				

Approval / Provisional Approval / Rejection of Change:

Director General ERA:______________________________________Date:__________________

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The Ethiopian Roads Authority (ERA) wishes to thank the UK Government's Department for International Development (DFID) through their Africa Community Access Programme (AFCAP) for their support in developing this *Pavement Design Manual Volume I Flexible Pavements - 2013*. The manual will be used by all authorities and organisations responsible for the provision of roads in Ethiopia.

From the outset, the approach to the development of the manual was to include all sectors and stakeholders in Ethiopia. The input from the international team of experts was supplemented by our own extensive local experience and expertise. Local knowledge and experience was shared through a series of meetings of Peer Review Groups comprising specialists drawn from within the local industry, which were established to provide advice and comments in their respective areas of expertise. The contribution of the Peer Review Group participants is gratefully acknowledged.

The final review and acceptance of the document was undertaken by an Executive Review Group. Special thanks are given to this group for their assistance in reviewing the final draft of the document. Finally, ERA would like to thank Crown Agents for their overall management of the project

This *Pavement Design Manual Volume I Flexible Pavements - 2013* is based on a review of the design standards of several countries. Most chapters are based closely on the Transport Research Laboratory Overseas Road Note 31, *A Guide to the Structural Design of Bitumen-Surfaced Roads in Tropical and Sub-Tropical Countries*. This reference document and companion TRL documents have drawn on the experience of TRL and collaborating organizations in numerous tropical and sub-tropical countries, including Ethiopia.

For the design of asphalt surfacings (Chapter 8), a subject that has given problems in the past, TRL's Overseas Road Note 19 '*A guide to the design of hot mix asphalt in tropical and sub-tropical countries'* has been the principle source document. This is based on many years of research experience in improving hot asphalt mix design in tropical environments with vehicle overloading and high temperature issues to take into account.

The main reference source for Chapter 9 is also a TRL manual namely Overseas Road Note 3: *A Guide to Surface Dressing in Tropical and Sub-Tropical Countries*.

Other major reference sources include AASHTO, and, in particular, the AASHTO Guide for Design of Pavement Structures, as revised in 1993. Asphalt Institute publications were reviewed for asphalt concrete and other hot-mix types. South African publications were also reviewed to assist in the development of a design well suited for the eastern African region in general and Ethiopia in particular.

As with the other manuals of this series, the intent was, where possible, and in the interests of uniformity, to use those tests and specifications included in the AASHTO and/or ASTM Materials references. Where no such reference exists for tests and specifications mentioned in this document, other references are used. Appendix K provides a complete list of test methods.

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GLOSSARY OF TERMS

Aggregate	Hard mineral elements of construction material mixtures, for example: sand, gravel (crushed or uncrushed) or crushed rock.
Asphalt	In American literature asphalt is another term for bitumen. The term is also commonly used in this way in Ethiopia. In other countries, asphalt is commonly used as shorthand for asphaltic concrete or, indeed, any design of high quality bitumen/aggregate mixture.
Asphalt Concrete	A mixture to predetermined proportions of aggregate, filler and bituminous binder material plant mixed and usually placed by means of a paving machine.
Asphalt Surfacing	The layer or layers of asphalt concrete constructed on top of the roadbase, and, in some cases, the shoulders.
Average Annual Daily Traffic (AADT)	The total yearly traffic volume in both directions divided by the number of days in the year.
Average Daily Traffic (ADT)	The total traffic volume during a given time period in whole days greater than one day and less than one year divided by the number of days in that time period.
Base Course	This is the main component of the pavement contributing to the spreading of the traffic loads. In many cases, it will consist of crushed stone or gravel, or of good quality gravelly soils or decomposed rock. Bituminous base courses may also be used (for higher classes of traffic). Materials stabilised with cement or lime may also be contemplated.
Binder Course	The lower course of an asphalt surfacing laid in more than one course.
Bitumen	The most common form of bitumen is the residue from the refining of crude oil after the more volatile material has been distilled off. It is essentially a very viscous liquid comprising many long-chain organic molecules. For use in roads it is practically solid at ambient temperatures but can be heated sufficiently to be poured and sprayed. Some natural bitumens can be found worldwide that are not distilled from crude oil but the amounts are very small in comparison.
Borrow Area	An area within designated boundaries, approved for the purpose of obtaining borrow material. A borrow pit is the excavated pit in a borrow area.
Borrow Material	Any gravel, sand, soil, rock or ash obtained from borrow areas, dumps or sources other than cut within the road prism and which is used in the construction of the specified work for a project. Not including crushed stone or sand obtained from commercial sources.
Boulder	A rock fragment, usually rounded by weathering or abrasion, with an

average dimension of 0.30 m or more.

Bound Pavement Materials	Pavement materials held together by an adhesive bond between the materials and another binding material such as bitumen.
Camber	The convexity given to the curved cross-section of a roadway.
Capping Layer	(Selected or improved subgrade). The top of embankment or bottom of excavation prior to construction of the pavement structure. Where very weak soils and/or expansive soils (such as black cotton soils) are encountered, a capping layer is sometimes necessary. This consists of better quality subgrade material imported from elsewhere or subgrade material improved by stabilisation (usually mechanical), and may also be considered as a lower quality sub-base.
Carriageway	That portion of the roadway including the various traffic lanes and auxiliary lanes but excluding shoulders.
Contraction Joint	A joint normally placed at recurrent intervals in a rigid slab to control transverse cracking.
Cross-Section	A vertical section showing the elevation of the existing ground, ground data and recommended works, usually at right angles to the centreline.
Crossfall	The difference in level measured transversely across the surface of the roadway.
Culvert	A structure, other than a bridge, which provides an opening under the carriageway or median for drainage or other purposes.
Cutting	Cutting shall mean all excavations from the road prism including side drains, and excavations for intersecting roads including, where classified as cut, excavations for open drains.
Chippings	Stones used for surface dressing (treatment).
Deformed Bar	A reinforcing bar for rigid slabs conforming to “Requirements for Deformations” in AASHTO Designations M 31M.
Design Period	The period of time that an initially constructed or rehabilitated pavement structure will perform before reaching a level of deterioration requiring more than routine or periodic maintenance.
Diverted Traffic	Traffic that changes from another route (or mode of transport) to the project road because of the improved pavement, but still travels between the same origin and destination.
Dowel	A load transfer device in a rigid slab, usually consisting of a plain round steel bar. Unlike a tie bar, a dowel may permit horizontal movement.

Equivalent Standard Axles (ESAs) A measure of the potential damage to a pavement caused by a vehicle axle load expressed as the number of 8.2 metric tonnes single axle loads that would cause the same amount of damage. The ESA values of all the traffic are combined to determine the total design traffic for the design period.

Equivalency Factors Used to convert traffic volumes into cumulative equivalent standard axle loads.

Equivalent Single Axle Load (ESA) Summation of equivalent 8.16 ton single axle loads used to combine mixed traffic to calculate the design traffic loading for the design period.

Escarpment Escarpments are geological features that are very steep and extend laterally for considerable distances, making it difficult or impossible to construct a road to avoid them. They are characterised by more than 50 five-metre contours per km and the transverse ground slopes perpendicular to the ground contours are generally greater than 50%.

Expansion Joint A joint located to provide for expansion of a rigid slab without damage to itself, adjacent slabs, or structures.

Fill Material of which a man-made raised structure or deposit such as an embankment is composed, including soil, soil-aggregate or rock. Material imported to replace unsuitable roadbed material is also classified as fill.

Flexible Pavements Includes primarily those pavements that have a bituminous (surface dressing or asphalt concrete) surface. The terms "flexible and rigid" are somewhat arbitrary and were primarily established to differentiate between asphalt and Portland cement concrete pavements.

Formation Level Level at top of subgrade.

Generated Traffic Additional traffic which occurs in response to the provision of improvement of the road.

Grading Modulus (GM) Related to the cumulative percentages by mass of material in a representative sample of aggregate, gravel or soil retained on the 2.36 mm, 0.425 mm and 0.075 mm sieves;

$$GM = 3 - \left(\frac{P_{2.36} + P_{0.425} + P_{0.075}}{100} \right)$$

where: $P_{2.36}$ = percentage passing 2.36 mm sieve

$P_{0.425}$ = percentage passing 0.425 mm sieve

$P_{0.075}$ = percentage passing 0.075 mm sieve

Heavy Vehicles Those having an unloaded weight of 3000 kg or more.

Hot mix asphalt (HMA)	This is a generic name for all high quality mixtures of aggregates and bitumen that use the grades of bitumen that must be heated in order to flow sufficiently to coat the aggregates. It includes Asphaltic Concrete, Dense Bitumen Macadam and Hot Rolled Asphalt.
Longitudinal Joint	A joint normally placed between traffic lanes in rigid pavements to control longitudinal cracking.
Maintenance	Routine work performed to keep a pavement as nearly as possible in its as-constructed condition under normal conditions of traffic and forces of nature.
Mountainous (Terrain)	Terrain that is rugged and very hilly with substantial restrictions in both horizontal and vertical alignment. It is defined as having 26-50 five-metre contours per km. The transverse ground slopes perpendicular to the ground contours are generally above 25%.
Normal Traffic	Traffic which would pass along the existing road or track even if no new pavement were provided.
Overlay	One or more courses of asphalt construction on an existing pavement. The overlay often includes a levelling course, to correct the contour of the old pavement, followed by a uniform course or courses to provide needed thickness.
Pavement Layers	The layers of different materials which comprise the pavement structure.
Project Specifications	The specifications relating to a specific project, which form part of the contract documents for such project, and which contain supplementary and/or amending specifications to the standard specifications.
Pumping	The ejection of foundation material, either wet or dry, through joints or cracks, or along edges of rigid slabs resulting from vertical movements of the slab under traffic.
Quarry	An area within designated boundaries, approved for the purpose of obtaining rock by sawing or blasting.
Reconstruction	The process by which a new pavement is constructed, utilizing mostly new materials, to replace an existing pavement.
Recycling	The reuse, usually after some processing, of a material that has already served its first-intended purpose.
Rehabilitation	Work undertaken to significantly extend the service life of an existing pavement. This may include overlays and pre overlay repairs, and may include complete removal and reconstruction of the existing pavement, or recycling of part of the existing materials.
Reinforcement	Steel embedded in a rigid slab to resist tensile stresses and detrimental opening of cracks.

Rigid Pavement A pavement structure which distributes loads to the subgrade having, as the main load bearing course, a Portland cement concrete slab of relatively high-bending resistance.

Roadbase A layer of material of defined thickness and width constructed on top of the sub-base, or in the absence thereof, the subgrade. A roadbase may extend to outside the carriageway.

Road Bed The natural in situ material on which the fill, or in the absence of fill, any pavement layers, are to be constructed.

Road Bed Material The material below the subgrade extending to such depth as affects the support of the pavement structure.

Road Prism That portion of the road construction included between the original ground level and the outer lines of the slopes of cuts, fills, side fills and side drains. It does not include sub-base, roadbase, surfacing, shoulders, or existing original ground.

Roadway The area normally travelled by vehicles and consisting of one or a number of contiguous traffic lanes, including auxiliary lanes and shoulders.

Rolling (Terrain) Terrain with low hills introducing moderate levels of rise and fall with some restrictions on vertical alignment. Defined as terrain with 11-25 five-metre contours per km. The transverse ground slopes perpendicular to the ground contours are generally between 3 and 25%.

Side Fill That portion of the imported material within the road prism which lies outside the fills, shoulders, roadbase and sub-base and is contained within such surface slopes as shown on the Drawings or as directed by the Engineer. A distinction between fills and side fill is only to be made if specified.

Side Drain Open longitudinal drain situated adjacent to and at the bottom of cut or fill slopes.

Stabilisation The treatment of the materials used in the construction of the road bed material, fill or pavement layers by the addition of a cementitious binder such as lime or Portland Cement or the mechanical modification of the material through the addition of a soil binder or a bituminous binder. Concrete and asphalt shall not be considered as materials that have been stabilised.

Sub-base The layer of material of specified dimensions on top of the subgrade and below the roadbase. The secondary load-spreading layer underlying the base course. Usually consisting of a material of lower quality than that used in the base course and particularly of lower bearing strength. Materials may be unprocessed natural gravel, gravel-sand, or gravel-sand-clay, with controlled gradation and plasticity characteristics. The sub-base also serves as a separating

	layer preventing contamination of the base course by the subgrade material and may play a role in the internal drainage of the pavement.
Subgrade	The surface upon which the pavement structure and shoulders are constructed. It is the top portion of the natural soil, either undisturbed (but recompacted) local material in cut sections, or soil excavated in cut or borrow areas and placed as compacted embankment.
Subsurface Drain	Covered drain constructed to intercept and remove subsoil water, including any pipes and permeable material in the drains.
Surface Treatment	The sealing or resealing of the carriageway or shoulders by means of one or more successive applications of bituminous binder and crushed stone chippings.
Surfacing	This comprises the top layers(s) of the flexible pavement and consists of a bituminous surface dressing or one or two layers of premixed bituminous material (generally asphalt concrete). Where premixed materials are laid in two layers, these are known as the wearing course and the binder course.
Tie Bar	A deformed steel bar or connector embedded across a joint in a rigid slab to prevent separation of abutting slabs.
Traffic Lane	Part of a travelled way intended for a single stream of traffic in one direction, which has normally been demarcated as such by road markings.
Traffic Volume	Volume of traffic usually expressed in terms of average annual daily traffic (AADT).
Typical Cross-Section	A cross-section of a road showing standard dimensional details and features of construction.
Unbound Pavement Materials	Naturally occurring or processed granular material which is not held together by the addition of a binder such as cement, lime or bitumen.
Wearing Course	The top course of an asphalt surfacing or, for gravel roads, the uppermost layer of construction of the roadway made of specified materials.
Welded Wire Fabric	Welded steel wire fabric for concrete reinforcement.

ABBREVIATIONS

AADT	Average Annual Daily Traffic
AASHO	American Association of State Highway Officials (previous designation)
AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Concrete
ACV	Aggregate Crushing Value – a measure of aggregate strength
ASTM	American Society for Testing Materials
BS	British Standard
CBR	California Bearing Ratio (as described in AASHTO T 193)
CRCP	Continuously Reinforced Concrete Pavement
DCP	Dynamic Cone Penetrometer
m_2, m_3	Drainage coefficients. Factors used to modify *layer coefficients* in flexible pavements to take account of climate, the effectiveness of internal pavement drainage and moisture sensitivity.
ERA	Ethiopian Road Authority
ESA	Equivalent standard axles. A measure of the damaging effect of vehicle axles.
FWD	Falling Weight Deflectometer
GM	Grading Modulus
HMA	Hot Mixed Asphalt
ICL	Initial Consumption of Lime test
IRI	International Roughness Index
LAA	Los Angeles Abrasion Value – a measure of aggregate strength.
MDD	Maximum Dry Density
NDT	Non destructive test
JPCP	Jointed Plain Concrete Pavement
JRCP	Jointed Reinforced Concrete Pavement
a_1, a_2, a_3	Strength coefficients. The empirical strength coefficients used for weighting the contribution of each layer of the pavement to

	the overall structural number (SN). They are modified by the drainage coefficients (see above).
NDT	Non destructive testing
PCC	Portland cement concrete
PMS	Pavement management system
RRD	Representative rebound deflection
S1 to S6	Subgrade strength classes used to characterize the subgrade in pavement design.
SN and MSN	Structural Number and Modified Structural Number. An index of overall pavement strength based on the thicknesses and strengths of each pavement layer.
SN_{eff} and MSN_{eff}	Effective Structural Number of an existing pavement.
T1 to T8	Traffic classes used to characterize the anticipated traffic in terms of ESA for flexible pavement design purposes.
h_1, h_2, h_3	Thicknesses of pavement surface, base and sub-base layers (existing or required).
TRL	Transport Research Laboratory, UK (formerly TRRL)
TRRL	Transport and Road Research Laboratory, UK
VOC	Vehicle Operating Costs
VFB	Voids Filled with Bitumen
VIM	Voids in the Mix
VMA	Voids in the Mineral Aggregate

1 INTRODUCTION

1.1 General

This manual gives recommendations for the structural design of 'flexible' pavements in Ethiopia. The definition of a flexible pavement is simply a pavement that *does not* include a layer of high strength concrete. Thus 'flexible pavements' include pavements with unbound granular aggregate layers and pavements with aggregate layers that are bound together with bitumen. It also includes pavements that may contain layers of aggregate that are bound together (or stabilised) with hydraulic binders such as cement and lime, but with relatively low levels of binder.

Pavements which include a layer of high strength Portland cement concrete are called 'rigid' pavements and are designed on different principles. The design of rigid pavements is treated separately in the ERA's *Pavement Design Manual Volume II Rigid Pavements.*

Gravel or 'unpaved' roads are also a form of flexible construction. Their design is similar to that of other flexible structures but the gravel itself wears away, depending on traffic, rainfall and terrain, hence additional material is required to make sure that the gravel is always thick enough. The design of gravel roads is dealt with in ERA's *Design Manual for Low Volume Roads.*

The design of rehabilitation for worn out flexible pavements and the design of strengthening overlays is covered in ERA's *Pavement Rehabilitation and Asphalt Overlay Manual.*

The manual is intended for engineers responsible for the design of new road pavements and is appropriate for roads which are required to carry up to 80 million cumulative equivalent standard axles in one direction. This upper limit is suitable at present for the most heavily trafficked roads in Ethiopia.

1.2 Principles

Road pavements are designed to limit the stress created at the subgrade level by the traffic travelling on the pavement surface so that the subgrade is not subject to significant deformations. The pavement spreads the concentrated loads of the vehicle wheels over a sufficiently large area at subgrade level. At the same time, the pavement materials themselves should not deteriorate to any serious extent within a specified period of time.

However, it is inevitable that road pavements will deteriorate with time and traffic, therefore, the goal of pavement design is to limit, during the period considered, the deterioration which affects the riding quality of the road, such as rutting, cracking, potholes and other such surface distresses, to acceptable levels.

At the end of the design period, a strengthening overlay would normally be required but other remedial treatments, such as major rehabilitation or reconstruction, may be needed. The design method aims at producing a pavement which will reach a relatively low level of deterioration at the end of the design period, assuming that routine and periodic maintenance are performed during that period.

An 'acceptable' riding quality depends on a match between what the users expect and what the highway authority (and hence the government) is prepared to provide. For roads with high traffic levels designed with high geometric standards (and higher vehicle speeds as a consequence), less distress will be expected or be considered acceptable. Hence trunk and link roads are expected to offer a higher standard of rideability than lower standards of road carrying lower levels of traffic such as collector roads, access roads etc.. These differences are implicitly considered in the design, although in broad terms rather than in precise, measurable economic terms.

To give satisfactory service, a flexible pavement must therefore resist the deterioration caused by the various deterioration mechanisms that are at work. These are the effects of traffic and the effects of the environment (essentially the effects of water and temperature).

The principal structural requirements are as follows and illustrated in Figure 1-1:

(1) The subgrade should be able to sustain traffic loading without excessive deformation; this is controlled by the vertical compressive stress or strain at this level.

(2) Bituminous materials and cement-bound materials used in roadbase design should not crack under the influence of traffic; this is controlled by the horizontal tensile stress or strain at the bottom of the bound layer.

(3) The roadbase is often the main structural layer of the pavement, required to distribute the applied traffic loading so that the underlying materials are not overstressed. It must be able to sustain the stress and strain generated within itself without excessive or rapid deterioration of any kind.

(4) In pavements containing bituminous materials, the internal deformation of these materials must be limited.

(5) The load spreading ability of granular sub-base and capping layers must be adequate to provide a satisfactory construction platform.

When some of the above criteria are not satisfied, distress or failure will occur. For instance, rutting may be the result of excessive internal deformation within bituminous materials, or excessive deformation at the subgrade level (or within granular layers above).

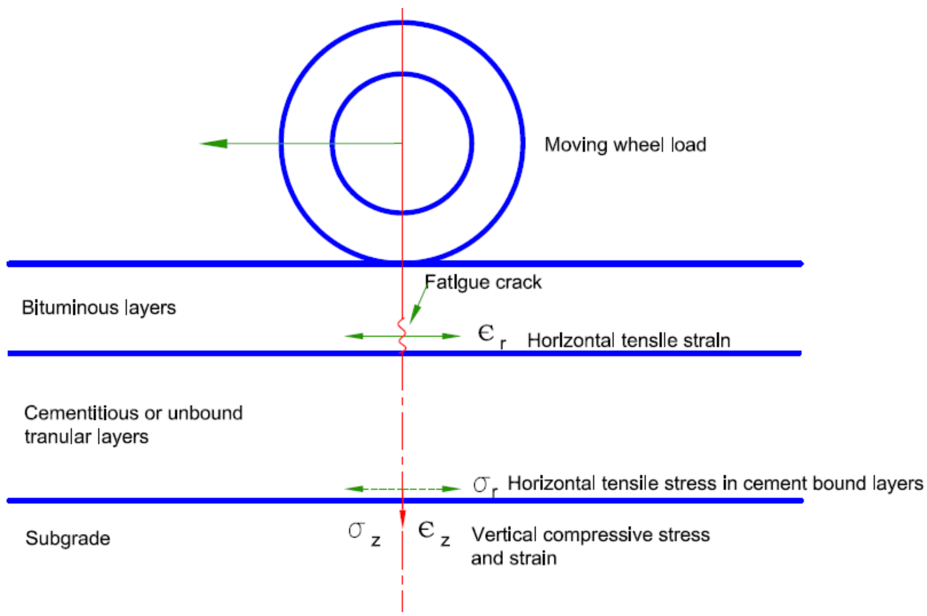


Figure 1-1 Critical Stresses and Strains in a Flexible Pavement Considered in Current Design Methods

1.3 Design Process

The main steps involved in designing a new road pavement are shown in Figure 1.2:

1. Surveying the possible route (usually part of the feasibility study).

2. Estimating the traffic in terms of the cumulative number of equivalent standard axles that will use the road over the selected design life (Chapter 2).

3. Characterizing the strength of the subgrade soil over which the road is to be built (Chapter 3).

4. Selecting pavement materials (Chapters 6, 7, 8 and 9).

5. Using the input data obtained in steps 1 to 4 to select a suitable structure from the catalogue of pavement structures presented in Chapter 10.

Intermediate chapters of the manual (Chapters 4 and 5) give guidance and background information related to the soils, shoulder design, drainage, and cross section assumptions underlying the design of the structures presented. The overall layout of the manual is shown in Figure 1.3.

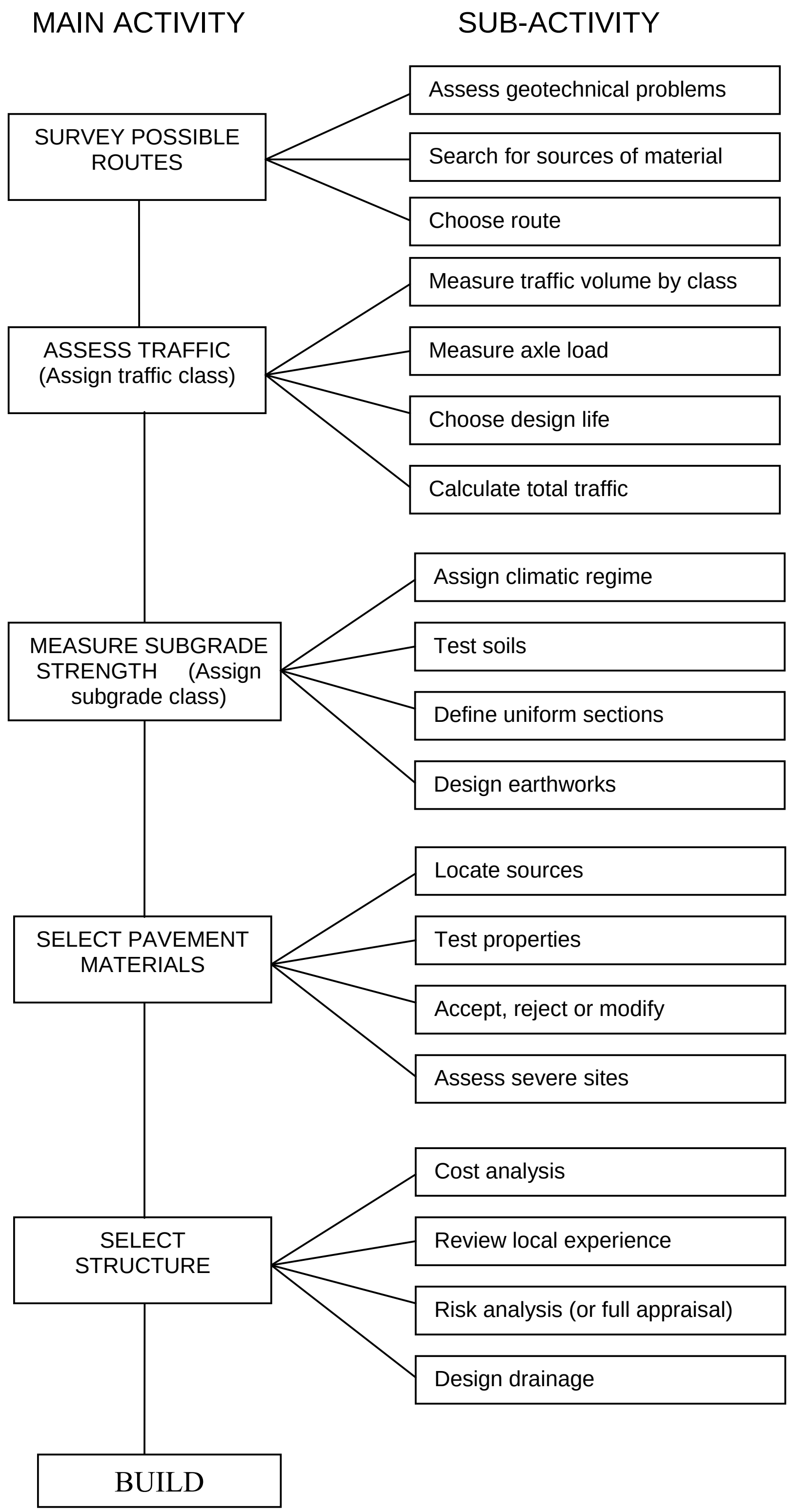


Figure 1-2 Pavement Design Process

1.4 Variability and Reliability

1.4.1 Traffic

Pavement design relies on knowledge of the expected level of traffic. Axle load studies (to determine equivalent axle loads) and traffic counts (to determine initial traffic volumes) are essential for a reliable design, together with estimates of traffic growth (Chapter 2). Yet traffic forecasting remains a difficult and often uncertain task. The parameters are rarely well known, particularly the axle loads and the projected growth. Although every effort must be made to reduce the uncertainty inherent to these estimates, caution is still recommended and some conservatism is justified. Moreover, sensitivity analyses of the resulting pavement structures to these parameters are recommended.

1.4.2 Climate

Climate also has a strong influence on the pavement performance, and may be accounted for in the design to some extent. This is particularly true for Ethiopia where a wide range of climatic zones are encountered; from desert in the north-east triangle around Djibouti, to temperate and mountainous (sub-alpine) over a significant part of the country, with annual rainfall up to 1500 mm.

The climate influences the subgrade moisture content and strength (Chapter 3) and requires precautions to ensure adequate drainage (Chapter 5). The rainfall also influences the selection of adequate pavement materials, such as the allowable limits of materials properties (Chapter 6), and is a potential incentive to use stabilised materials (Chapter 7).

The temperature influences the selection and design of bituminous surfacings (Chapters 8 and 9).

Climate also affects the nature of the soils and rocks encountered at subgrade level. Soil-forming processes are very active and the surface rocks are often deeply weathered. The soils themselves occasionally display unusual properties which can pose considerable problems for road designers.

1.4.3 Materials and construction

Variability in material properties and construction control is generally much greater than desired by the design engineer and must be taken into account explicitly in the design process. Only a very small percentage of the surface of a road needs to show distress for the road to be considered unacceptable by road users. It is therefore the weakest parts of the road that show distress first which are important in design; it is ***not*** the average values of road strength that is important.

In well-controlled full-scale experiments the variability in strength is such that the ten per cent of the road which performs best will carry about six times more traffic before reaching a defined terminal condition than the ten per cent which performs least well. Under normal construction conditions this spread of behaviour becomes even greater.

Much of this variability can be explained through the variability of the main factors that directly affect performance. For example, the strength of the subgrade varies from place to place along the road, both laterally and longitudinally and this can be measured. Also the strength and thickness of the pavement layers varies from place to place and typical values for existing roads can also be measured (e.g. with a DCP). Therefore, if the likely variability is known beforehand, it is possible, in principle, for it to be taken into account in design. *It is false economy to minimise the extent of preliminary investigations to determine this variability*.

In practice it is the variability of subgrade strength that is most important. All other factors are controlled by means of specifications i.e. by setting minimum acceptable values for the key properties. However, even when the variability of subgrade strength and pavement material properties is taken into account, there often remains a considerable variation in performance between nominally identical pavements. Optimum design therefore remains partly dependent on knowledge of the performance of in-service roads and quantification of the variability of the observed performance itself. Thus there is always scope for improving designs based on local experience.

It is the task of the designer to estimate likely variations in layer thicknesses and material strengths so that realistic target values and tolerances can be set in the specifications to ensure that satisfactory road performance can be guaranteed as far as is possible. The thickness and strength values described in this manual are essentially minimum values but practical considerations require that they are interpreted as lower ten percentile values with 90 per cent of all test results exceeding the values quoted. Random variations in thickness and strength should be such that minor deficiencies in thickness or strength do not occur concomitantly, or very rarely so. Good construction practices to ensure this randomness and also to minimize variations themselves cannot be over emphasized.

1.5 Basis for the Design Catalogue

In view of the statistical nature of pavement design caused by the large uncertainties in traffic forecasting and the variability in material properties, climate and road behaviour, the design charts (Chapter 10) are presented as a catalogue of structures. Each structure is applicable over a range of traffic and subgrade strength. Such a procedure makes the charts easy to use, but it is important that the designer is conversant with the notes applicable to each chart.

The pavement designs are based largely on the results of full-scale experiments and studies of the performance of as-built existing road networks. However, pavement designs are now required that are reliable up to relatively high traffic levels and empirical evidence for such designs is much less than for the structures for low and intermediate levels of traffic. In order to derive such designs, judicious use has been made of theoretical (or mechanistic) analysis techniques. The details of this are shown in Appendix H. There are many problems with such methods and it is thought by many experts that they generally result in relatively conservative designs. In deriving the designs in this manual, care has been taken to minimize this problem.

Another method of providing confidence in the extended designs is to compare them with designs for similar conditions obtained by using methods developed in countries with a strong history of pavement research such as the USA, Australia and RSA. This has been done, however, the same problem still applies namely that there is much less evidence for the performance of very heavily trafficked roads simply because there are very few of them.

1.6 Economic Considerations

The pavement design engineer, on the basis of the site investigations, should ascertain that materials required for all components of the pavement structure are available. This task should be performed concurrently with the design discussed in the following chapters since, for a given traffic and subgrade conditions, several structures are offered. Hence, the availability of materials will often influence or dictate the choice between the alternate pavement structures.

Next, the prevailing unit costs of the materials should be compiled, based either on recent works of similar type and magnitude in the vicinity of the proposed project, or by an analysis of the mobilization, production and haulage costs.

In the past the selection of pavement type has been based simply on the lowest construction cost on the assumption that the structures in the design catalogue for a particular traffic level will all last as long as each other. Whilst this is true in principle, different structures deteriorate in different ways and require differing levels of maintenance. Furthermore some structures are more tolerant of poor maintenance and are therefore likely to perform better if maintenance cannot be carried out in a timely manner. The point here is that in the long term, and depending on the local circumstances, the best choice of pavement structure can vary between those in the catalogue and will not necessarily be the cheapest to construct. It is therefore good practice to carry out a life cycle cost analysis, anticipating maintenance needs and risk factors and including them in the analysis. In Ethiopia at the present time there is insufficient experience of many of the possible structures and their likely maintenance requirements for this to be done with much accuracy. However data is being accumulated to assist with this in future and more roads need to be built using alternative structures. In the meantime use could be made of models such as HDM 4 to investigate options, although care is required because such models must be calibrated to local conditions; the lack of data is an impediment to accurate analysis.

Eventually it is also recommended that road user or vehicle operating costs are also included in the life cycle cost analysis. This is because vehicle operating costs are the highest costs in a transport system and depend on the surface condition or roughness of the road. Smoother roads will save vehicle operating costs, which include fuel costs and other costs that affect the national economy.

While researching the recent unit costs of particular materials, knowledge of past experience with these materials should necessarily develop, and their performance can be evaluated. This experience can, in turn, be incorporated into the process of selection of the materials.

Vehicle operating costs depend on the condition of the road surface. The road surface deterioration, hence its condition, depends on the nature of the traffic, the properties of the pavement layers materials, the environment, and the maintenance strategy adopted. Knowledge of the interaction between these factors is the object of ERA’s Pavement Management System (PMS) and is expected to evolve and be refined as the PMS procedures are implemented in Ethiopia. Ideally, in the future it will be possible to design a road in such a way that, provided maintenance and strengthening can be carried out at the proper time, the total cost of the road, i.e. the sum of construction costs, maintenance costs and road user costs, can be minimized. As road condition surveys and PMS procedures are conducted on a regular basis, additional information will be collected to allow road performance models to be refined. Pavement structural design and pavement rehabilitation design may then become an integral part of the management system in which design could be modified according to the expected maintenance inputs in such a way that the most economic strategies could be adopted. These refinements lie in the future, but research in this domain has been used, in part, in preparing the recommendations presented in this manual.

For the pavement structures recommended in this manual, the level of deterioration that is reached by the end of the design period should be limited to levels which yield acceptable economic designs under most anticipated conditions. Routine and periodic maintenance activities are assumed to be performed at a reasonable and not excessive level.

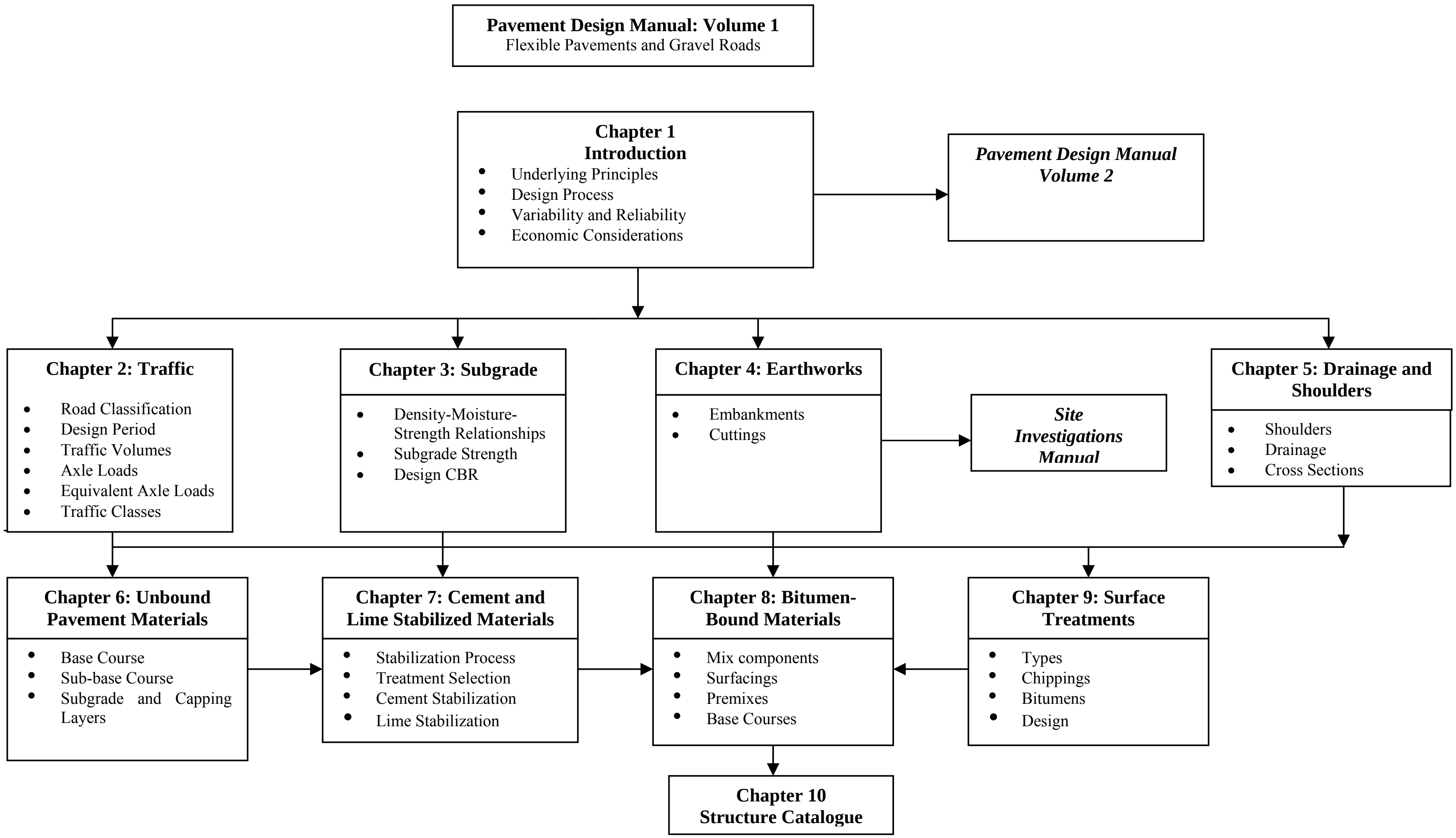


Figure 1-3 Organization of Manual

2 TRAFFIC

2.1 General

The deterioration of paved roads caused by traffic results from both the magnitude of the individual wheel loads and the number of times these loads are applied. It is necessary to consider not only the total number of vehicles that will use the road but also the wheel loads (or, for convenience, the axle loads) of these vehicles. Equivalency factors are used to convert traffic volumes into cumulative standard axle loads and this is discussed in this section. For paved roads, traffic classes are defined by ranges of cumulative number of equivalent standard axles (ESAs).

The process by which traffic is evaluated is illustrated in Figure 2-1. A complete design example of traffic calculations for flexible pavement design is presented in Section 2.7.

2.2 Design Period

Determining an appropriate design period is the first step towards pavement design. Many factors may influence this decision, including budget constraints. However, the designer should follow certain guidelines in choosing an appropriate design period, taking into account the conditions governing the project. Some of the points to consider include:

i) Functional importance of the road
ii) Traffic volume
iii) Location and terrain of the project
iv) Financial constraints
v) Difficulty in forecasting traffic

Usually it is economical to construct roads with longer design periods for important roads and for roads with high traffic volume. Where rehabilitation would cause major inconvenience to road users, a longer period may be used. For roads in difficult locations and terrain where regular maintenance proves to be costly and time consuming because of poor access and non-availability of nearby construction material sources, a longer design period is also appropriate.

Difficulties in traffic forecasting may also influence the design period. When accurate traffic estimates cannot be made, it may be advisable to reduce the design period to avoid costly overdesign and to adopt a stage construction strategy to cater for unexpected traffic growth.

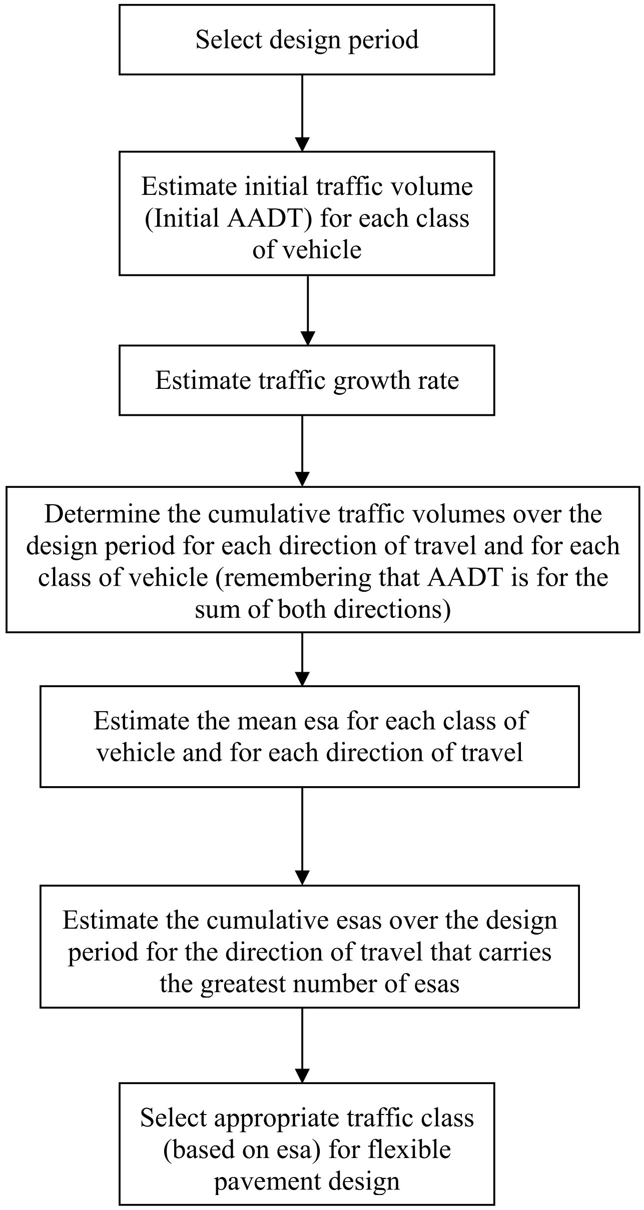


Figure 2-1 Traffic Evaluation

Bearing in mind the above considerations, it is important that the designer consults ERA at the outset of the project to ascertain the design period. Table 2.1 shows the general guidelines:

Table 2-1 Design Period

Road Classification	Design Period (years)
Trunk Road	20
Link Road	20
Main Access Road	15
Other Roads	10

2.3 Traffic Volumes

2.3.1 Vehicle Classification

The types of vehicles are defined according to Table 2.2.

Table 2-2: Vehicle Classification

Class	Type	Axles	Description
1	Car	2	Passenger cars and taxis
2	Pick-up/4-wheel drive	2	Pick-up, minibus, Land Rovers, Land Cruisers
3	Small bus	2	≤ 27 seats
4	Bus/coach	2	> 27 seats
5	Small truck	2	≤ 3.5 tonnes
6	Medium truck	2 or 3	3.5 – 7.5 tonnes
7	Large 2-axled truck	2	> 7.5 tonnes
8	3-axled truck	3	>7.5 tonnes
9	4-axled truck	4	*
10	5-axled truck	5	*
11	6-axled truck	6	*
12	2-axled trailer	2	*
13	3-axled trailer	3	*

*Not needed for definition

Traffic data may be available based on a simpler classification system using only five classes namely Classes 1, 2 and 3 combined, Class 4, Classes 5 and 6 combined, Classes 7 to 11 combined and a class for articulated trucks. However, every effort should be made to carry out a more detailed traffic classification as shown in Table 2.2 because the average ESA values for the heavy vehicle classes can be very different; this more detailed classification will enable a more accurate estimate of the total ESA values to be obtained.

2.3.2 Initial Traffic Volumes

In order to determine the total traffic over the design life of the road, the first step is to estimate initial traffic volumes. The estimate should be the (Annual) Average Daily Traffic (AADT) currently using the route (or, more specifically, the AADT expected to use the route during the first year the road is placed in service) classified into the thirteen classes of vehicles described above. Adjustments will usually be required between the AADT based on the latest traffic counts and the AADT during the first year of service. These adjustments can be made using the growth factors discussed below.

Based on the review of various traffic studies conducted in Ethiopia in recent years, it has been concluded that traffic volumes are very erratic and do not show any identifiable general trends. This makes it difficult to predict growth rates and future volumes. However, it is expected that, as traffic increases, the variability will decrease and it will become a little easier to forecast traffic more accurately.

The AADT is defined as the total annual traffic summed for ***both*** directions and divided by 365. It is usually obtained by recording actual traffic volumes over a shorter period from which the AADT is then estimated. It should be noted that for structural design purposes the traffic loading in ***one*** direction is required and for this reason care is always required when interpreting AADT figures. For long projects, large differences in traffic along the road may make it necessary to estimate the flow at several locations.

Traffic counts carried out over a short period as a basis for estimating the AADT can produce estimates which are subject to large errors because traffic volumes can have large daily, weekly, monthly and seasonal variations. The daily variability in traffic volume depends on the volume of traffic, with particularly high variability on roads carrying less than 1000 vehicles per day. Traffic volumes vary more from day-to-day than from week-to-week over the year. Thus there are large errors associated with estimating annual AADTs from traffic counts of only a few days duration, or excluding the weekend. For the same reason there is a rapid decrease in the likely error as the duration of the counting period increases up to one week. For counts of longer duration, improvements in accuracy are less pronounced. Traffic volumes also vary from month-to-month (seasonal variation), so that a weekly traffic count repeated at intervals during the year provides a better base for estimating the annual volume of traffic than a continuous traffic count of the same total duration. Traffic also varies considerably through a 24-hour period and this needs to be taken into account explicitly as outlined below.

Based on the above, and in order to reduce error, it is recommended that traffic counts to establish AADT at a specific site conform to the following practice:

The counts are for seven consecutive days.

i) The counts on some of the days are for a full 24 hours with, preferably, at least one 24-hour count on a weekday and one during a weekend. On the other days 16-hour counts should be sufficient. These should be extrapolated to 24-hour values in the same proportion as the 16-hour/24-hour split on those days when full 24-hour counts have been undertaken.
ii) Counts are avoided at times when travel activity is abnormal for short periods due to the payment of wages and salaries, public holidays, etc. If abnormal traffic flows persist for extended periods, for example during harvest times, additional counts need to be made to ensure this traffic is properly included.
iii) If possible, the 7-day counts should be repeated several times throughout the year.

Countrywide traffic data should, preferably, be collected on a systematic basis to enable seasonal trends in traffic volumes to be quantified. The frequency of counting shown in the following Table is recommended.

Table 2-3: Frequency of Traffic Counts

Road Classification	7-day Traffic Counts
Trunk Road	Quarterly
Link Road	Quarterly
Main Access Road	Every 6 months
Other Roads	Every 3 years

2.3.3 Traffic Forecast

Even with stable economic conditions, traffic forecasting is an uncertain process. Although the pavement design engineer may often receive help from specialised professionals at this stage of the traffic evaluation, some general remarks are in order.

To forecast traffic growth it is usually necessary to separate traffic into the following three categories:

(a) Normal traffic. Traffic which would pass along the existing road or track even if no new pavement were provided.

(b) Diverted traffic. Traffic that changes from another route (or mode of transport) to the project road because of the improved pavement, but still travels between the same origin and destination.

(c) Generated traffic. Additional traffic which occurs in response to the provision or improvement of the road.

Normal traffic

The most common method of forecasting normal traffic is to extrapolate data on traffic levels and assume that growth will either remain constant in absolute terms i.e. a fixed number of vehicles per year, or constant in relative terms (a fixed percentage increase). As a general rule it is only safe to extrapolate forward for as many years as reliable traffic data exist from the past, and for as many years as the same general economic conditions are expected to continue.

As an alternative, growth can be related linearly to anticipated growth of Gross Domestic Product (GDP). This is normally preferable since it explicitly takes into account changes in overall economic activity.

If it is thought that a particular component of the traffic will grow at a different rate to the rest (e.g. a category of trucks, due to the development of an industry) it should be specifically identified and dealt with separately; a uniform growth rate among the various traffic classes should not necessarily be assumed a priori.

Whatever the forecasting procedure used, it is essential to consider the realism of forecast future levels.

Diverted traffic

Where parallel routes exist, traffic will usually travel on the quickest or cheapest route although this may not necessarily be the shortest. Thus, surfacing an existing road may divert traffic from a parallel and shorter route because higher speeds are possible on the surfaced road. Origin and destination surveys should preferably be carried out to provide data on the traffic diversions likely to arise.

Analysis of origin / destination survey data can be done using computer based programs to determine the diverted traffic volumes.

Diversion from other transport modes, such as rail or water, is not easy to forecast. Transport of bulk commodities will normally be by the cheapest mode, though this may not be the quickest.

Diverted traffic is normally forecast to grow at the same rate as traffic on the road from which it diverted.

Generated traffic

Generated traffic arises either because a journey becomes more attractive by virtue of a cost or time reduction or because of the *increased* development that is brought about by the road investment. Generated traffic is also difficult to forecast accurately and can be easily overestimated.

The recommended approach to forecasting generated traffic is to use demand relationships which show how traffic increases as the cost of a journey decreases. Studies carried out in similar countries give an average for the price elasticity of demand for transport of about -1.0. This means that a one per cent decrease in transport costs leads to a one per cent increase in traffic. This is extra traffic over and above the increase in *normal traffic*.

At this stage, the designer has the required elements to determine the initial and forecast AADT. For paved roads, it is still necessary to consider the axle loads in order to determine the cumulative equivalent standard axle loads (ESA) over the design period (see Section 2.4 below) in order to select an appropriate traffic class (Section 2.4).

Determination of Cumulative Traffic Volumes

In order to determine the cumulative number of vehicles over the design period of the road, the following procedure should be followed:

1. Determine the initial traffic volume, $AADT(m)_0$, of each traffic class (m) using the results of the traffic survey and any other recent traffic count information that is available.
2. Estimate the annual growth rate "*i*" expressed as a decimal fraction, and the anticipated number of years "n" between the traffic survey and the opening of the road.
3. For each vehicle class, estimate the traffic in the first year that the road is opened to traffic. For normal traffic this is given by

$$AADT(m)_1 = AADT(m)_0\,(1+i)^n \qquad \text{Equation 2.1}$$

4. For each vehicle class, add the estimate for diverted traffic and for generated traffic if any are anticipated.

For structural pavement design the cumulative traffic loading of each of the motorised vehicle classes over the design life of the road in one direction is required. For a given class, m, this is given by the following equation:

$$T(m) = 0.5 \times 365 \times AADT(m)_0\,[(1+i/100)^N - 1]/(i/100) \qquad \text{Equation 2.2}$$

Where

$T(m)$	=	the cumulative traffic of traffic class m
$AADT(m)_1$	=	The AADT of traffic class m in the first year
N	=	the design period in years
i	=	the annual growth rate of traffic in percent

The cumulative traffic for each class of vehicle is multiplied by the average number of equivalent standard axles of vehicles in that class to calculate the cumulative total number of equivalent standard axles over the life of the road.

2.4 Axle Loads

2.4.1 Axle equivalency

The damage that vehicles do to a paved road is highly dependent on the axle loads of the vehicles. For pavement design purposes the damaging power of axles is related to a “standard” axle of 8.16 metric tons using empirical equivalency factors. In order to determine the cumulative axle load damage that a pavement will sustain during its design life, it is necessary to express the total number of heavy vehicles that will use the road over this period in terms of the cumulative number of equivalent standard axles (ESAs).

Axle loads can be converted and compared using standard factors to determine the damaging power of different vehicle types. A vehicle’s damaging power, or Equivalency Factor (EF), can be expressed as the number of equivalent standard axles (ESAs), in units of 80kN. The design lives of pavements are expressed in terms of the ESAs they are designed to carry.

2.4.2 Axle Load Surveys

Axle load surveys must be carried out to determine the axle load distribution of a sample of the heavy vehicles using the road (vehicles in Class 4 and Classes 6 to 13, Table 2-2). Data collected from these surveys are used to calculate the mean number of ESA for a vehicle in each class. These values are then used in conjunction with traffic forecasts to determine the predicted cumulative equivalent standard axles that the road will carry over its design life.

Most countries have regulations for controlling the size and weight of vehicles using the roads. However, in many, including Ethiopia, enforcement has proved difficult and, as a consequence, vehicles are often grossly overloaded. Not only are the numbers of overloaded vehicles very high but the magnitude of their overloads is also very high. Examples have been reported where axle loads over 100 per cent higher than those permitted in the regulations have been observed and where axle loads have exceeded the range of portable weighbridges designed for the purpose of weighing vehicles. In such cases, a pavement designer who assumes that the vehicles are conforming to the country’s regulations on vehicle weight and axle loading will design roads that will fail very early in their expected lives.

No regular axle load surveys are conducted in Ethiopia at present hence each individual project depends on its own axle load survey data. Such surveys are for a limited time period and may not provide reliable and representative data unless some care is taken in selecting when the surveys are carried out and how they are conducted.

Ideally, several surveys at periods that will reflect seasonal changes in the magnitude of axle loads are recommended. Portable vehicle-wheel weighing devices are available which enable a small team to weigh up to 90 vehicles per hour.

The duration of the survey should be based on the same considerations as for traffic counting outlined in Section 2.3.

On certain roads it may be necessary to consider whether the axle load distribution of the traffic travelling in one direction is the same as that of the traffic travelling in the opposite direction. Significant differences between the two streams can occur on roads serving ports, quarries, cement works, etc., where the vehicles travelling one way are heavily loaded but are empty on the return journey. In such cases the results from the more heavily trafficked lane should be used when converting volumes to ESA for pavement design. Similarly, special allowance must be made for unusual axle loads on roads which mainly serve one specific economic activity, since this can result in a particular vehicle type being predominant in the traffic spectrum. This is often the case, for example, in such areas as timber extraction or mining areas.

It should be noted that vehicle owners and operators will quickly alter their behaviour if an axle load survey is underway. They will, for example, divert to another route, if such a route is available, or stop their overloading practices until the axle load survey is completed. Thus a typical survey will show that axle loads will begin to decrease steadily within half a day of its commencement and after three or four days most vehicles will be legally loaded. Thus a good strategy is to carry out several very short surveys (say 2 or 3 days) rather than one long one.

Countrywide axle load data should, preferably, be collected on a systematic basis to enable trends to be quantified and to provide data for road design purposes. The frequency of axle load surveys shown in the following Table is recommended.

Table 2-4: Frequency of Axle Load Surveys

Road Classification	**2 or 3-day Axle Load Surveys**
Trunk Road	Quarterly
Link Road	Quarterly
Main Access Road	Every 6 months
Other Roads	Every 3 years

Once the axle load data has been gathered, the mean equivalency factor for each class of vehicle must be calculated. Computer programs may be used to assist with the analysis of the results from axle load surveys. Such programs provide a detailed tabulation of the survey results and determine the mean equivalency factors for each vehicle type if required. Alternatively, standard spreadsheet programs can be used.

The following method of analysis is recommended:

1. Determine the equivalency factors for each of the wheel loads measured during the axle load survey, using Table 2-5 or the accompanying equation, in order to obtain the equivalency factors for vehicle axles. The factors for the axles are added together to obtain the equivalency factor for each of the vehicles.
2. For vehicles with multiple axles (tandems, triples etc.), each axle in the multiple group is considered separately. Although the exact ESA values for multiple axles have been shown to differ from this, the differences are dependent on road structure and axle spacing but are relatively small compared with the problem of uneven distribution of load between the axles in the multiple sets. Thus treating the axles separately provides the most reliable estimate of road damage.

3. Determine the mean equivalency factor for each class of heavy vehicle travelling in each direction. It is customary to assume that the axle load distribution of the heavy vehicles will remain unchanged for the design period of the pavement.

This method of determining the mean equivalency factors must always be used; calculating the equivalency factor for the average axle load is incorrect and leads to very large errors.

2.4.3 Equivalent standard axles per vehicle class

The number of equivalent standard axles (ef) of an axle is related to the axle load as follows:

$$ef = (L/8160)^n \quad \text{(for loads in kg)} \qquad \text{Equation 2.3}$$

or

$$ef = (L/80)^n \quad \text{(for loads in kN)} \qquad \text{Equation 2.4}$$

Where:

ef	=	number of equivalent standard axles (ESAs)
L	=	axle load (in kg or kN)
n	=	damage exponent (n = 4.5).

Table 2.3 shows the values of ESA for different axle loads.

The sum of the individual *ef* values for each axle of the vehicle gives the equivalence factor for the vehicle as a whole, EF(*m*). Guidance on the likely average EF(*m*) for different vehicle classes derived from historical data is given in Table 2.6. However, data from any recent axle load survey on the road in question or a similar road in the vicinity is better than using countrywide averages.

2.4.4 Cumulative Equivalent Standard Axles over the Design Period

The cumulative ESAs over the design period for each vehicle class is obtained by multiplying EF(*m*) by the cumulative traffic, T(*m*). The total number of cumulative standard axles for all vehicle classes is then obtained by adding together the values of EF(*m*) x T(*m*) for all the classes.

In some cases there will be distinct differences in each direction and separate vehicle damage factors for each direction should be derived. The higher of the two directional values should be used for design.

On narrow roads the traffic tends to be more channelised than on wider two-lane roads. In such cases the effective traffic loading is greater than that for a wider road and the design traffic loading is calculated using the relationships given in Table 2.7.

The pavement design thicknesses required for the design lane are usually applied to the whole carriageway width.

Table 2-5 Equivalency Factors for Different Axle Loads (Flexible Pavements)

Wheel load (10^3 kg)	Axle load (10^3 kg)	Equivalence factor
1.5	3	0.01
2.0	4	0.04
2.5	5	0.11
3.0	6	0.25
3.5	7	0.50
4.0	8	0.91
4.5	9	1.55
5.0	10	2.50
5.5	11	3.93
6.0	12	5.67
6.5	13	8.13
7.0	14	11.3
7.5	15	15.5
8.0	16	20.7
8.5	17	27.2
9.0	18	35.2
9.5	19	44.9
10.0	20	56.5

Note: The equivalency factors given in Table 2-5 are to be used solely for flexible pavement design. Refer to Volume 2 for specific factors for rigid pavements

Table 2-6 Average equivalency factors for different vehicle types

Class	Type	No of axles	Average ESA per vehicle - all loaded	Average ESA per vehicle - half loaded[(1)]
1	Car	2	-	-
2	4-wheel drive	2	-	-
3	Minibus	2	0.3	0.15
4	Bus/coach	2	2.0	1.0
5	Small truck/PU	2	1.5	0.7
6	Medium truck	2	5	2.5
7	Large 2-axled truck	2	10	5
8	3-axled truck	3	12	3.5
9	4-axled truck	4	15	7.5
10	5-axled truck	5	17	8.5
11	6-axled truck	6	17	8.5
12	2-axled trailer	2	10	5
13	3 or 4-axled trailer	3/4	12	6

Note It is common to find that vehicles have no back-load hence half the vehicles are likely to be empty, or nearly so.

Table 2-7 Factors for design traffic loading

Cross Section	Paved width	Corrected design traffic loading (ESA)	Explanatory notes
Single carriageway	< 3.5 m	Double the sum of ESAs in both directions	The driving pattern on this cross-section is highly channelized.
	Min. 3.5 m but less than 4.5.m	The sum of ESAs in both directions	Traffic in both directions uses the same lane
	Min. 4.5 m but less than 6 m	80% of the ESAs in both directions	To allow for overlap in the centre section of the road
	6 m or wider	Total ESAs in the heaviest loaded direction	Little traffic overlap in the centre section of the road.
More than one lane in each direction		90% of the total ESAs in the studied direction	The majority of vehicles use one lane in each direction.

2.5 Traffic Classes for Flexible Pavement Design

Accurate estimates of cumulative traffic are difficult to achieve due to errors in the surveys and uncertainties with regard to traffic growth, axle loads and axle equivalencies. To a reasonable extent, however, pavement thickness design is not very sensitive to cumulative axle loads and the method recommended in this manual provides fixed structures of paved roads for ranges of traffic as shown in Table 2.8.

Table 2-8 Traffic Classes for Flexible Pavement Design

Traffic Classes	Range of ESAs (millions)
LV1	< 0.01
LV2	0.01 – 0.1
T1/LV3 (see note)	0.1 – 0.3
T2/LV4 (see note)	0.3 – 0.5
T2/LV5 (see note)	0.5 – 0.7
T3	0.7 – 1.5
T4	1.5 – 3.0
T5	3.0 – 6.0
T6	6.0 – 10
T7	10 – 17
T8	17 - 30
T9	30 - 50
T10*	50 - 80
T11	>80

Notes. There are more options available for the Low Volume classes which use granular unbound road bases and sub-bases (i.e. Chart A). These are dealt with in the ERA *LVR Design Manual*.

*T10 is suitable for traffic up to 80 mesas. At this level the pavement is expected to be ‘long-life and suitable for higher traffic levels.

As long as the estimate of cumulative equivalent standard axles is close to the centre of one of the ranges, any errors are unlikely to affect the choice of pavement design. However, if estimates of cumulative traffic are close to the boundaries of the traffic ranges, then the basic traffic data and forecasts should be re-evaluated and sensitivity analyses carried out to ensure that the choice of traffic class is appropriate. Depending on the degree of accuracy achieved, if in doubt, selecting the next higher traffic class may be appropriate.

2.6 Design Example

Initial traffic volumes in terms of AADTs have been established for a section of a trunk road under study, as follows:

Table 2-9 Initial Traffic Volumes (Example)

Vehicle	Class	AADT
Car	1	250
Bus	4	40
Truck	6	130
Trucks with trailers	7 and 12	180

Note. For simplicity only 4 traffic classes have been used

The anticipated traffic growth is a constant 5%, and the opening of the road is scheduled for 3 years from now. In addition, an axle load survey has been conducted, giving representative axle loads for the various classes of heavy vehicles, such as given below for truck-trailers (it is assumed that the loads are equally representative for each direction of traffic). Equivalency factors for the sample of truck-trailers, and a mean equivalency factor for that class of heavy vehicles, can be calculated as shown:

Table 2-10 Equivalency Factors (Example)

Truck No.	Axle 1		Axle 2		Axle 3		Axle 4		Total factor
	Load (kg)	Factor	Load (kg)	Factor	Load (kg)	Factor	Load (kg)	Factor	
1	6780	0.43	14150	11.91	8290	1.07	8370	1.12	14.5
2	6260	0.3	12920	7.91	8090	0.96	9940	2.43	11.6
3	6350	0.32	13000	8.13	8490	1.2	9340	1.84	11.5
4	5480	0.17	12480	6.77	7940	0.88	9470	1.95	9.8
5	6450	0.35	8880	1.46	6290	0.31	10160	2.68	4.8
6	5550	0.18	12240	6.2	8550	1.23	10150	2.67	10.3
7	5500	0.17	11820	5.3	7640	0.74	9420	1.91	8.1
8	4570	0.07	13930	11.1	2720	0.01	2410	0.00	11.2
9	4190	0.05	15300	16.92	3110	0.01	2450	0.00	17
10	4940	0.1	15060	15.76	2880	0.1	2800	0.01	15.9
Mean equivalency factor for truck-trailers									11.5

This Table should signal a warning to the design engineer. Of the 10 trucks weighed, none appear to be empty or nearly so. In principle there is no need to actually weigh empty trucks but their numbers need to be counted and, ideally, they should be recorded in the field sheets so that there is no danger that the calculated average esa values will be incorrect. There is no evidence in this Table that the empty trucks have been incorporated in the calculation correctly. Because of this problem it is often recommended that *all* trucks are weighed (or a random sample if traffic is high). This eliminates any concerns that only loaded trucks have been weighed. For the sake of this example, it is assumed that the average esa figure is correct and that no empty trucks use this road.

In this example it is assumed that similar calculations have been performed, giving mean equivalency factors for buses and trucks of 0.14 and 6.67 respectively.

The projected AADTs in three years can be calculated as (current AADTs) x $(1.05)^3$ and the corresponding one-directional volumes for each class of vehicle in three years are:

Table 2-11 ADDT and Flow (Example)

Vehicle	Class	AADT in 3 years	One directional flow
Car	1	290	145
Bus	4	46	23
Truck	6	150	75
Trucks with trailers	7 and 12	208	104

Selecting a design period of 20 years, the cumulative number of vehicles in one direction over the design period is calculated as follows:

Table 2-12 Cumulative Traffic (Example)

Vehicle	Class	Cumulative number in one direction over 20 years
Car	1	365 x 145 x $[(1.05)^{20}-1]/0.05$ = 1,750,016
Bus	4	365 x 23 x $[(1.05)^{20}-1]/0.05$ = 277,589
Truck	6	365 x 75 x $[(1.05)^{20}-1]/0.05$ = 905,180
Trucks with trailers	7 and 12	365 x 104 x $[(1.05)^{20}-1]/0.05$ = 1,255,184

Finally, the cumulative numbers of ESAs over the design period are calculated as follows, using the cumulative numbers of vehicles previously calculated and the equivalency factors:

Table 2-13 Cumulative ESAs (Example)

Vehicle	Class	Cumulative number of vehicles	Equivalency factor	Total ESA (10^6)
Car	1	1,750,016	0	0.0
Bus	4	277,589	0.14	0.0
Truck	6	905,180	6.67	6.0
Truck-trailer	7 and 12	1,255,184	11.47	14.4
			Total ESAs	20.4

Based on the above analysis, the trunk road under study belongs to the traffic class T8 for flexible pavement design.

3 SUBGRADE

3.1 General

The type of subgrade soil is largely determined by the location of the road. However, where the soils within the possible corridor for the road vary significantly in strength from place to place, it is desirable to locate the pavement on the stronger soils if this does not conflict with other constraints. For this reason, amongst others, the pavement engineer should be involved in the route selection process.

The strength of the road subgrade for flexible pavements is commonly assessed in terms of the California Bearing Ratio (CBR) and this is dependent on the *type of soil,* its *density,* and its *moisture content*. Direct assessment of the likely strength or CBR of the subgrade soil under the completed road pavement is often difficult to make. Its value, however, can be inferred from an estimate of the density and equilibrium (or ultimate) moisture content of the subgrade together with knowledge of the relationship between strength, density and moisture content for the soil in question. This relationship must be determined in the laboratory. The density of the subgrade soil can be controlled within limits by compaction at a suitable moisture content at the time of construction.

The eventual moisture content of the subgrade soil is governed by the local climate, the depth of the water table below the road surface, and the provisions that are made for both internal and external drainage.

It is useful to recall some basic relationships relating soil strength, density and moisture content and how they affect the final subgrade strength (Section 3.2). In Section 3.3, the various steps leading to the selection of a design CBR are described.

3.2 Density-Moisture Content-Strength Relationships of the Subgrade

During road construction the (dry) density of the subgrade soil (and its moisture content) is modified from its original state by compaction at subgrade level (in cuts) and by compaction of the excavated materials used in embankments. The moisture content is adjusted in order to make it easier to achieve a high level of compaction.

Upon completion of the construction operations, the density of the compacted subgrade soil will remain approximately the same except for some residual compaction under traffic and possible volume variations of certain moisture sensitive soils. However the moisture content of the subgrade will change, depending on climate, soil properties, depth of water table, rainfall and drainage. It is knowledge of this condition of the subgrade that is required in the design process.

To illustrate the above discussion, Figures 3.1 and 3.2 illustrate examples of relationships between density, moisture content and CBR. Two figures are shown to emphasize that the relationships are specific to the nature of the subgrade soil. The figures indicate a ‘likely level of compaction achieved during construction’. It can be seen from the figures that, as the moisture content increases at constant density (moving to the right) the CBR decreases quite quickly. If the soil becomes saturated, i.e. the air voids become filled with water and decrease to zero, the soil becomes very weak indeed.

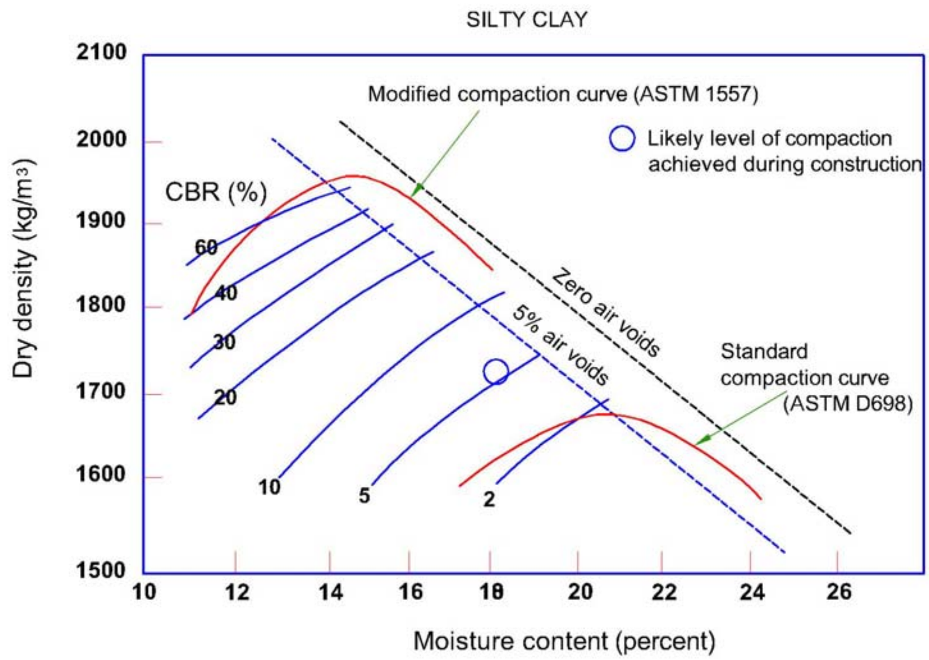


Figure 3-1 Dry Density, Moisture Content, Soil Strength Relationship for a Silty Clay

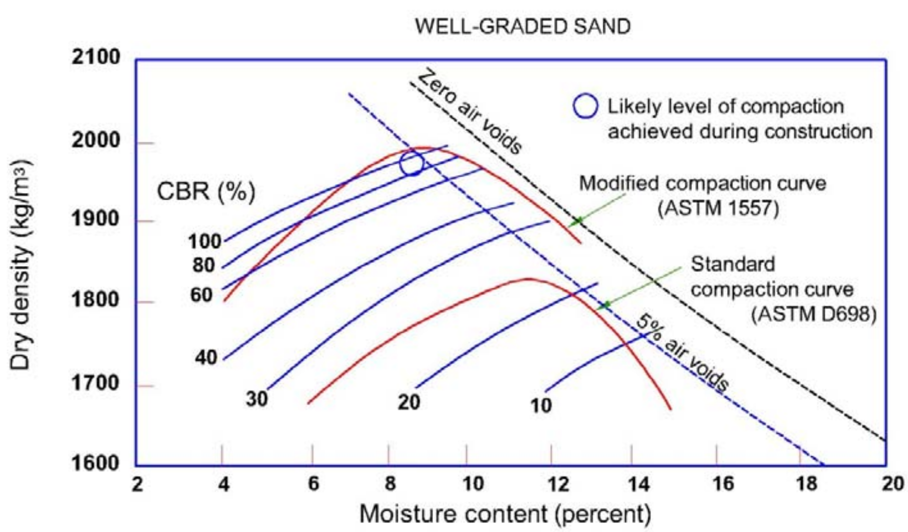


Figure 3-2 Dry Density, Moisture Content, Soil Strength Relationship for a Well-Graded Sand

3.3 Design Subgrade Strength

To determine the subgrade strength for the design of the road pavement, it is necessary to first determine the density-moisture content-strength relationship(s) specific to the subgrade soil(s) encountered along the road under study. It is then necessary to select the density which will be representative of the subgrade once compacted and to estimate the subgrade moisture content that will ultimately govern the design, i.e. the moisture content after construction.

3.3.1 Estimated Design Moisture Content of the Subgrade

After the pavement is constructed, the moisture content of the subgrade will generally change. In the dry southeast and northeast parts of Ethiopia, a decrease in the moisture content may be expected. The moisture content can increase elsewhere due to perched water tables during wet seasons. In low-lying areas, the normal water table may be close to the finished subgrade level and influence the ultimate moisture content. In areas with deep water tables and proper design and construction, it is less likely that the subgrade will get wetter after construction.

To simplify the estimation of subgrade strength for design, three classifications of subgrade conditions are defined.

Category 1: The water table is high enough during the rainy season to govern the moisture content.

The depth at which the water table becomes the dominant factor depends on the type of soil. For example, in non-plastic soils the water table will dominate the subgrade moisture content only when it rises to within 1 m of the road surface; in sandy clays (PI<20 %) the water table will dominate when it rises to within 3m of the road surface; and in heavy clays (PI>40 %) the water table will dominate when it rises to within 7m of the road surface.

For this category it is best to observe the water table in boreholes and determine its seasonal high. When it has been established that the road is in a Category 1 zone, it is necessary to determine the moisture content at the wettest/weakest likely condition.

The expected moisture content may be measured below existing pavements in the vicinity, if such pavements exist. Care should be taken to make the measurements when the water table is at its highest level. These pavements should be greater than 3m wide and more than two years old and samples should preferably be taken from under the carriageway about 0.5m from the edge. Allowance can be made for different soil types by virtue of the fact that the ratio of subgrade moisture content to plastic limit is the same for different subgrade soils when the water table and climatic conditions are similar.

In some cases, such as in particularly low-lying areas or where it is determined (or strongly suspected) that the water table is close to the subgrade finished level, it is appropriate to consider whether the moisture content will reach or approach saturation. The design strength must then be based on this assumption by testing samples compacted to the target density at optimum moisture content but then tested after a period of soaking.

Category 2: The water table is deep, but rainfall can influence the subgrade moisture content under the road.

These conditions occur when rainfall exceeds evaporation and transpiration for at least two months of the year. The rainfall in such areas, which represent the greater part of Ethiopia, is greater than 250 mm per year and is seasonal. The moisture condition under an impermeable pavement will depend on the balance between the water entering the subgrade (e.g. through the shoulders and at the edges of the pavement) during wet weather and the moisture leaving the ground by evapo-transpiration during dry periods. The moisture condition for design purposes can be taken as the optimum moisture content given by ASTM Test Method D 698.

Exceptions to this situation are when perched water tables are suspected, or where there may be doubts about keeping the pavement surface sufficiently waterproof and ensuring adequate internal drainage (see Chapter 5). In these latter cases it will be prudent to consider saturated conditions.

Category 3: Deep water table and arid climate.

These conditions occur where the climate is dry throughout most of the year with annual rainfall of 250 mm or less. They may therefore be encountered in the low altitude areas of the northeast (areas of the Tigray, Welo and Harerge regions) and in the southeast (Harerge and Bale regions).

In such conditions, the moisture content is likely to be relatively low. It is recommended that for design purposes a value of 80% of the optimum moisture content obtained by ASTM Test Method D 698, reflecting the probability that the subgrade will lose some moisture and gain strength after construction.

The methods of estimating the subgrade moisture content for design outlined above are based on the assumption that the road pavement is virtually impermeable. Dense bitumen-bound materials, stabilised soils with only very fine cracks, and crushed stone or gravel with more than 15 per cent of material finer than the 75 micron sieve are themselves impermeable (permeability less than 10^{-7} metres per second) and therefore subgrades under road pavements incorporating these materials are unlikely to be influenced by water infiltrating directly from above.

However, if water shed from the road surface or from elsewhere is able to penetrate to the subgrade for any reason, the subgrade may become much wetter. In such cases the strength of subgrades with moisture conditions in Categories 1 and 2 should be assessed on the basis of saturated CBR samples, as previously indicated. Subgrades with moisture conditions in Category 3 are unlikely to wet up significantly and the subgrade moisture content for design in such situations can be taken as the optimum moisture content given by ASTM Test Method D 698.

3.3.2 Representative Density

After estimating the subgrade moisture content for design, it is then necessary to determine a representative density at which a design CBR value will be selected.

To specify densities during construction, it is recommended that the top 250 mm of all subgrades be compacted to a relative density of at least 100% of the maximum dry density achieved by ASTM Test Method D 698 (light or standard compaction). Alternatively, at least 93% of the maximum dry density achieved by ASTM Test Method D 1557 may be specified. With modern compaction equipment, a relative density of 95% of the density obtained in the heavier compaction test should be achieved without difficulty, but tighter control of the moisture content will be necessary and considerable variability will remain. It is therefore recommended that design is based on the lower density and if a higher density is achieved it should be considered an additional factor of safety.

Variations from these usual assumptions are possible on a case by case basis, in the light of local experience and laboratory testing as outlined below and described in the Site Investigations Manual. It is vital to verify that the density assumed in design is consistent with the minimum density specified for a particular road project.

3.3.3 Specific Density-Moisture Content- Strength Relationships

The ERA *Site Investigation Manual* recommends standard compaction tests (ASTM D 698) to measure the CBR on samples moulded at 100% MDD and OMC (standard compaction). Each typical soil can be subjected to more detailed testing involving three levels of compaction, and, at each level, two conditions of moisture. The design CBR is then obtained by interpolation as illustrated below. This method enables an estimate to be made of the subgrade CBR at different densities and allows the effects of different levels of compaction control on the structural design to be evaluated.

3.3.4 Design CBR and Design Subgrade Strength Class

Figure 3.3 shows a detailed dry density/moisture content/CBR relationship for a sandy-clay soil that was obtained by compacting samples at several moisture contents to three levels of compaction. By interpolation, a design subgrade CBR of about 15 per cent is obtained if a relative density of 100 per cent of the maximum dry density obtained in the ASTM Test Method D 698 Test is specified and the subgrade moisture content was estimated to be 20 percent.

The procedure outlined above and also detailed in the ERA *Site Investigation Manual* is not as elaborate as to give complete curves as shown in Figure 3-3, but is nevertheless sufficient to conduct the necessary interpolations. This laboratory determination is the first (and generally preferred) option available to obtain a design CBR.

As an additional option (but not recommended for use on its own), in areas where existing roads have been built on the same subgrade, direct measurements of the subgrade strengths can also be made using a dynamic cone penetrometer (e.g. the TRL Dynamic Cone Penetrometer, see Appendix I). Except for direct measurements of CBR under existing pavements, in situ CBR measurements of subgrade soils are not recommended because of the difficulty of ensuring that the moisture and density conditions at the time of test are representative of those expected under the completed pavement.

The structural catalogue given in this manual requires that the subgrade strength for design be assigned to one of six strength classes reflecting the sensitivity of thickness design to subgrade strength. The classes are defined in Table 3.1. For subgrades with CBRs less than 2, special treatment is required.

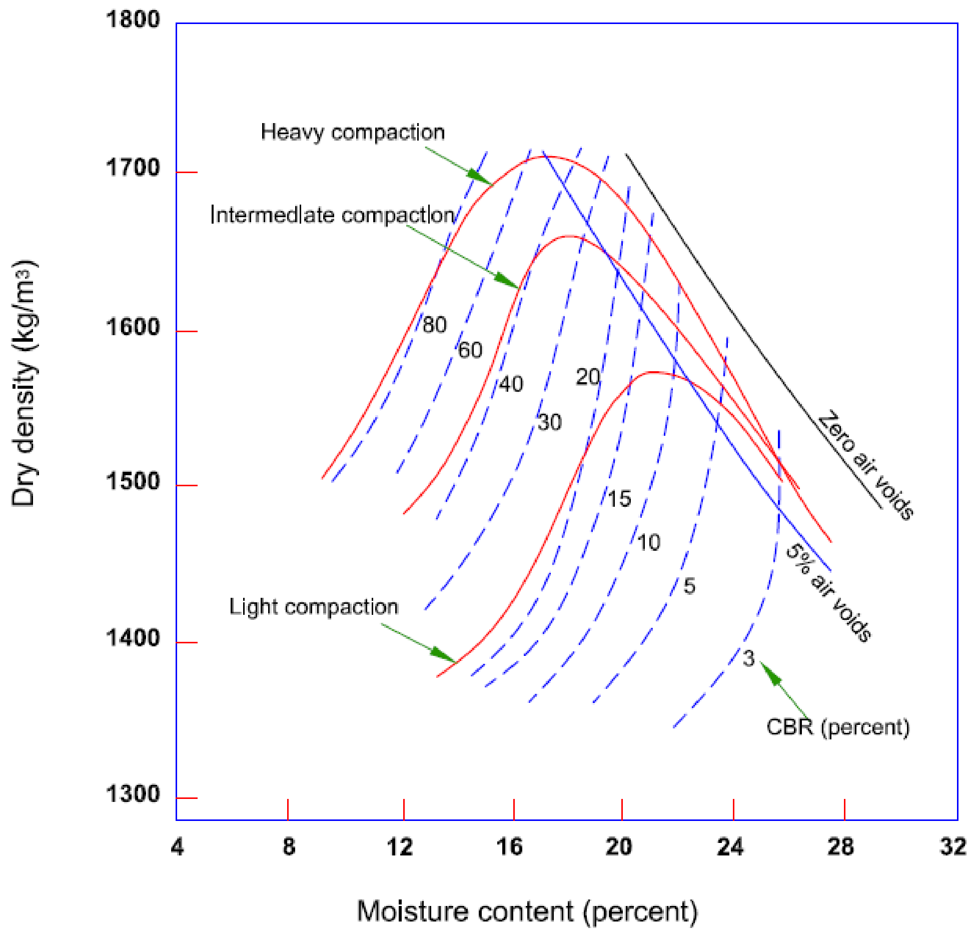


Figure 3-3 Dry Density, Moisture Content, CBR for Sandy-Clay Soil

Table 3-1 Subgrade Strength Classes

Class	CBR Range (%)
S1	<3
S2	3,4
S3	5,6,7
S4	8 - 14
S5	15 - 30
S6	>30

A less precise estimate of the minimum subgrade strength class can be obtained from Table 3.2. This table shows the estimated minimum strength class for five types of subgrade soil for various depths of water table, assuming that the subgrade is compacted to not less than 95 per cent of the maximum dry density attainable in the ASTM Test Method D 698 (Light Compaction). The table is appropriate for subgrade moisture Categories 1 and 2 but can be used for Category 3 if conservative strength estimates are acceptable.

Table 3-2 Estimated Design Subgrade Strength Class under Sealed Roads in the Presence of a Water Table

Depth of water table from formation level (m)	Subgrade strength class				
	Non plastic sand	Sandy clay PI = 10	Sandy clay PI =20	Silty clay PI = 30	Heavy clay PI = 40
0.5	S4	S4	S2	S2	S1
1	S5	S4	S3	S2	S1
2	S5	S5	S4	S3	S2
3	S6	S5	S4	S3	S2

Notes: 1 The highest seasonal level attained by the water table should be used.
2 Table 3-2 is not applicable for silt, micaceous, organic or tropically weathered clays. Laboratory CBR tests should be undertaken for these soils.

The design subgrade strength class together with the traffic class obtained in Chapter 2 are then used with the catalogue of structures to determine the pavement layer thicknesses (Chapter 10).

3.3.5 Delineation of Subgrade Areas

A road section for which a pavement design is undertaken should be subdivided into subgrade areas where the subgrade CBR can be reasonably expected to be uniform, without significant variations. Significant variations in this respect means variations that would yield different subgrade classes as defined above. However, it is not practical to create too many separate sections. The soils investigations should delineate subgrade design units on the basis of geology, pedology, drainage conditions and topography, and consider soil categories which have fairly consistent geotechnical characteristics (e.g. grading, plasticity, CBR). Usually, the number of soil categories and the number of uniform subgrade areas will not exceed 4 or 5 for a given road project unless the road is particularly long.

Generally, it is not advisable to define short design sections that are stronger than the adjacent sections. However, where there are short sections that are particularly weak, these are usually defined because it is uneconomic to use the design suitable for these sections over long sections that are much stronger. Thus it is important to differentiate between localized very poor soils and general subgrade areas. Normally, localized poor soils will be removed and replaced with suitable materials.

Where the subgrade CBR values are very variable, the design should consider the respective benefits and costs of short sections and of a conservative approach based on the worst conditions over longer sections.

Other useful correlations for assessing the subgrade strength include:

i) a correlation between the nature of the soils (as given in the Unified Soil Classification System, USCS, described in ASTM Method D2487) and typical design CBR values;

ii) the AASHTO classification.

By nature, these classifications cover all soils encountered in Ethiopia. The correlation between the nature of the soil and typical design CBR values is shown in Table 3-3.

The AASHTO classification is given in AASHTO M145. It includes seven basic groups (A-1 to A-7) and twelve subgroups. Of particular interest is the Group Index, which is used as a general guide to the load bearing ability of a soil. The group index is a function of the liquid limit, the plasticity index and the amount of material passing the 0.075mm sieve. Under average conditions of good drainage and thorough compaction, the supporting value of a material may be assumed an inverse ratio to its group index, i.e. a group index of 0 indicates a “good” subgrade material and a group index of 20 or more indicates a poor subgrade material.

Table 3-3 Typical Design CBR Values

<table>
<tr><th colspan="2">Major Divisions</th><th colspan="2">Symbol</th><th>Name</th><th>Value as Subgrade</th><th>Typical Design CBR Values</th></tr>
<tr><td rowspan="10">COARSE GRAINED SOIL</td><td rowspan="5">GRAVEL AND GRAVELLY SOILS</td><td colspan="2">GW</td><td>Well-graded gravels or gravel- sand mixtures, little or no fines</td><td>Excellent</td><td>40-80</td></tr>
<tr><td colspan="2">GP</td><td>Poorly graded gravels or gravel-sand mixtures, little or no fines</td><td>Good to excellent</td><td>30-60</td></tr>
<tr><td rowspan="2">GM</td><td>d</td><td rowspan="2">Silty gravels, gravel-sand-silt mixtures</td><td>Good to excellent</td><td>40-60</td></tr>
<tr><td>u</td><td>Good</td><td>20-30</td></tr>
<tr><td colspan="2">GC</td><td>Clayey gravels, gravel-sand-clay mixtures</td><td>Good</td><td>20-40</td></tr>
<tr><td rowspan="5">SAND AND SANDY SOILS</td><td colspan="2">SW</td><td>Well-graded sands or gravelly sands, little or no fines</td><td>Good</td><td>20-40</td></tr>
<tr><td colspan="2">SP</td><td>Poorly graded sands or gravelly sands, little or no fines</td><td>Fair to good</td><td>10-40</td></tr>
<tr><td rowspan="2">SM</td><td>d</td><td rowspan="2">Silty sands, sand-silt mixtures</td><td>Fair to good</td><td>15-40</td></tr>
<tr><td>u</td><td>Fair</td><td>10-20</td></tr>
<tr><td colspan="2">SC</td><td>Clayey sands, sand-clay mixtures</td><td>Poor to fair</td><td>5-20</td></tr>
<tr><td rowspan="6">FINE GRAINED SOILS</td><td rowspan="3">SILTS AND CLAYS LL IS LESS THAN 50</td><td colspan="2">ML</td><td>Inorganic silts and very fine sands, rock Flour, silty or clayey fine sands or clayey Silts with slight plasticity</td><td>Poor to fair</td><td>15 or less</td></tr>
<tr><td colspan="2">CL</td><td>Inorganic clays of low to medium plasticity, Gravelly clays, sandy clays, silty clays, Lean clays</td><td>Poor to fair</td><td>15 or less</td></tr>
<tr><td colspan="2">OL</td><td>Organic silts and organic silt-clays of low Plasticity</td><td>Poor</td><td>5 or less</td></tr>
<tr><td rowspan="3">SILTS AND CLAYS LL IS GREATER THAN 50</td><td colspan="2">MH</td><td>Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts</td><td>Poor</td><td>10 or less</td></tr>
<tr><td colspan="2">CH</td><td>Inorganic clays of high plasticity, fat clays</td><td>Poor to fair</td><td>15 or less</td></tr>
<tr><td colspan="2">OH</td><td>Organic clays of medium to high plasticity, Organic silts</td><td>Poor to very poor</td><td>5 or less</td></tr>
<tr><td colspan="2">HIGHLY ORGANIC SOILS</td><td colspan="2">Pt</td><td>Peat and other highly organic soils</td><td>Not suitable</td><td></td></tr>
</table>

NOTE: The division of GM and SM groups into subdivisions of d and u is on basis of Atterberg limits; suffix d (e.g. GMd) will be used when the liquid limit is 25 or less and the plasticity index is 5 or less; the suffix u will be used otherwise.

4 EARTHWORKS

4.1 Introduction

In this chapter guidelines are given which pertain to both the geotechnical design of the roadway and, more specifically, to pavement design. The geotechnical details given herein are not intended to replace a comprehensive geotechnical design. This is dealt with in ERA's *Geotechnical Design Manual*. The primary purpose of this chapter is to raise awareness of the problems that might be encountered with embankments and cuttings so that geotechnical investigations can be called for if required.

Some considerations relative to earthworks belong to the manual dealing with soils rather than the manual dealing with pavements. For this reason, this chapter should be read in conjunction with ERA's *Site Investigation Manual*.

4.2 Embankments

4.2.1 General

For the design of embankments, the following areas of concern should be addressed:

i) Foundations conditions, with their associated potential problems of settlements and stability.
ii) Embankment materials and related topics regarding specified methods of placing and compaction.
iii) Protection of the completed embankment slopes.

Potential problems in these areas should be identified during the reconnaissance, as described in ERA's *Site Investigation Manual*.

4.2.2 Embankment foundations

The design of embankments over soft and compressible soils requires the determination of both the magnitude of *settlement* which will occur under the future embankments and the anticipated rate of settlement. It also requires verification of the allowable height of embankment, or side slopes, or construction rate to prevent shear failure and ensure embankment *stability*.

Considerations regarding settlements and stability are covered separately for convenience, although it should be realized that the problems are usually concomitant. Also, some of the typical solutions (e.g. accelerated consolidation, removal of soft soils) often deal with both aspects of the problem.

4.2.3 Expansive Soils

Problem foundations for embankments in Ethiopia include expansive clays (black cotton soils). Expansive soils are those that exhibit particularly large volumetric changes, both shrinkage and swell, due to variations in their moisture content. They exhibit poor bearing capacity (similar to some stability problems).

Particular care is needed with such expansive soils and, if construction in these soils cannot be avoided, earthworks must be designed to minimize subsequent changes in moisture content and consequent volume changes. When the subgrade is a particularly expansive soil, it may be necessary to replace the expansive material with non-expansive impermeable soil to the depth affected by seasonal moisture changes. However, the measures to minimize the effect of expansive soils must be both economic and proportionate to the risk of pavement damage and increased maintenance costs.

Problems associated with construction over expansive soils are usually the seasonal changes in these soils rather than the low bearing strength. Expansive soils are often relatively strong at equilibrium moisture content. Distress occurs as seasonal wetting causes soils at the edge of the pavement to wet and dry out at rates differing from those further under the bituminous surfacing. This mechanism causes differential movements over the roadway cross section and associated crack development, beginning at the shoulder and proceeding towards the carriageway.

Mitigation measures include the excavation and replacement of expansive soils, although this may not be economically viable. Less expensive options include the provision of constant moisture contents over the full width of the carriageway, possibly through sealing of the shoulders, the replacement of the upper layer of the expansive soil, and provision of a minimum cover. During construction, the roadbed of expansive soil should be kept moist and covered with earthworks prior to any drying. Attempts to process and compact the soil beyond normal density requirements is not required. Fill material over the expansive soils shall be impermeable soils with a plasticity index of greater than 15%. Considerably more detail about dealing with expansive soils is provided in ERAs *Design Manual for Low Volume Roads.*

Side drains should be avoided in areas of expansive soils. Where this is not possible, they should be placed a minimum distance away from the toe of the side slope, as indicated in the *ERA Geometric Design Manual*. The side slope should also be reduced to a maximum 1:6.

4.2.4 Embankment Settlement

Settlement during construction is often unavoidable but post-construction settlement (i.e. after paving and opening to traffic) must be minimized. Differential settlements are the most detrimental to riding quality. In order to reduce these differential settlements, it is often convenient to set a limit to the total post-construction settlement, e.g. of the order of 3 to 5 cm.

Clays that have developed from volcanic ash may have a fragile structure which may be prone to collapse under embankment loads.

It is important to distinguish between two types of soft and compressible soils namely under-consolidated silt-clay mixtures and organic soils. The difference lies in their consolidation characteristics.

It is also convenient to group the possible solutions to settlement as follows:

- Methods involving excavation or displacement. Excavation may be partial or complete. Displacement may be by rock fill or controlled failure.
- Consolidation methods or combination of methods, including preloading, surcharging and accelerated vertical drainage (e.g. by prefabricated drains).

With under-consolidated silts and clays, the settlements occur mostly during the primary consolidation and the primary consolidation parameters govern, together with the thickness of the compressible soils. The consolidation parameters are determined from laboratory testing of undisturbed samples. It is important to obtain and preserve good quality samples to carry out reasonable predictions of settlement magnitude and rate. It is also important to verify whether stratified or varied deposits are present, as this can make horizontal drainage far more important than vertical drainage in the consolidation process. Vertical as well as horizontal drainage must be accounted for in the design of prefabricated (wick) drains. The use of sand drains, although efficient, is less common than in the past due to the difficulties and cost of installation.

Traditional methods of predicting settlements are given in the geotechnical literature. The choice between the methods of alleviating the problems will depend on the time available for construction and consolidation, and by stability concerns.

Strict specifications and monitoring of settlement (e.g. settlement platforms, piezometers) are often essential to the success of the design and the embankment performance.

Organic soils are a common cause of excessive post-construction settlement (i.e. affecting the rideability and potentially the structural integrity of the pavement), especially due to secondary settlement after the primary consolidation has taken place. In addition, their bearing capacity remains poor even when consolidated. It is therefore best to avoid such materials altogether during the selection of the route alignment. If this is not possible, the methods consisting of removing and replacing the organic soils are preferable. These methods may still not be feasible, either because the organic deposits are very thick, or because underground water flows should not be restricted. In such cases, traditional methods similar to those outlined above for under-consolidated silts-clays may be used. Very gentle side slopes or wide embankments with berms are often used under these conditions primarily for stability reasons, but low, wide embankments are occasionally used to limit the rate of settlement to acceptable levels. In addition, the use of geosynthetics (geogrids, geotextiles) is expanding to help in the construction, improve stability and reduce differential settlement. Geosynthetics are also used as reinforcement of the embankment itself.

4.2.5 Embankment stability

The design of embankments regarding their stability should be initiated by the verification that the weight of the embankment will not overcome the shear strength of the foundation soil (punching failure), with consideration given to the side slopes.

Further analyses includes verification of the safety factor (e.g. 1.2 or 1.3) against rotational failure (slip circles) or random shaped failure, using a variety of methods now made easier by computerized means. It is important when using such methods to consider failure modes that may be dictated by the local conditions (wedge shaped failures, sloping firm ground under the soft soils, etc.).

It is also useful to note that there are a variety of simplified methods and design charts which give a reasonable approximation of the safety factor under common conditions.

During construction of embankments over soft soils, pore water pressures can be monitored using piezometers. Further precautions can be taken by installing inclinometers to detect any movement of soil that might indicate that unstable conditions exist.

When no specific foundation problem is encountered, the suitability of the side slopes is largely determined by the internal stability of the embankment material (provided erosion is controlled). In those cases, general recommendations can be made as follows for embankments up to 10 metres high:

Table 4-1 Slope Ratio Table – Vertical to Horizontal

Material	Height of Slope (m)	Side Slope (V:H)		Back Slope
		Fill	Cut	
Earth Soil	0.0 – 1.0	1:3	1:2	1:3
	1.0 – 2.0	1:2		1:2
	>2.0	2:3		2:3
Strong Rock	0.0 – 2.0	4:5		2:1
	>2.0	1:1		4:1
Weathered Rock	0.0 – 2.0	2:3		2:1
	>2.0	1:1		3:1
Decomposed Rock	0.0 – 1.0	1:3		1:3
	2.0 – 2.0	1:2		1:2
	>2.0	2:3		2:3
Black Cotton Soil (expansive clays)(1)	0.0 – 2.0	1:6	-	-
	>2.0	1:4		

1 Move ditch away from fill

This Table should be used as a guide only, particularly because applicable standards in rock cuts are highly dependent on costs.

Certain soils that may be present at subgrade level may be unstable at 1:2 side slopes and therefore a higher standard will need to be applied for these soils. In particularly wet areas of Ethiopia, it may be desirable to use flatter slopes when the embankments are silty or clayey.

Steeper slopes in combination with reinforcement of the embankment material may be necessary in certain urban sites.

Slope configuration and treatments in areas with identified slope stability problems should be addressed as a final design issue.

4.2.6 Types of Embankment Materials

Embankment fill material normally comes from adjacent cut sections. If the quantities are insufficient, borrow areas are required, preferably adjacent to the road. If the quality is not suitable, additional haulage will be required.

Most soils are suitable for embankment construction and the use of the majority of available materials should be encouraged. Some soils are, however, generally unsuitable:

- Materials with more than 5% by weight of organic materials
- Materials with a swell of more than 3% (e.g. black cotton soils)
- Clays with a plasticity index over 45 or a liquid limit over 90

Exceptions may be made to the above on a case by case basis. For instance, when alternatives are prohibitively expensive, black cotton soils may be used provided methods to alleviate their associated problems are effected (see *Site Investigation Manual* and the *Design Manual for Low Volume Roads*).

Rock fill may be used to form the base of the embankments in uniform layers not exceeding 1 metre in thickness (oversize materials to be reduced in size). Voids in the top layer (30 cm) of rock should be filled. Rock in embankments should be 600 mm or more below the finished subgrade/top of embankment.

Soils with lower plasticity should be preferred for the lower layers, and dried as necessary to allow proper compaction. The best materials should be reserved for the upper layers of the subgrade.

4.2.7 Placing and Compaction of Embankment Materials

When the embankment is to be placed and compacted on hillsides, or when a new embankment is to be compacted against existing embankments, or when the embankment is to be built a portion at a time, the slope against which the embankment is to be placed should be benched continuously as the embankment is brought up in layers. This applies whenever the slopes against which the embankment is to be constructed are steeper than 1 (V) to 3 (H). Benching should be a minimum of 2 metres in width in order to integrate the new embankment with the existing slope. Material cut out should be recompacted along with the new embankment.

A uniform compaction is important in order to prevent uneven settlement. Some settlement can be tolerated, but it should be minimized, particularly at the approaches to bridges and culverts where adequate compaction is essential.

It is usual, unless otherwise indicated in special provisions, to specify that a minimum density must be achieved. It is therefore essential that laboratory tests be carried out to determine the dry density/moisture content relationships for the soils to be used and to define the achievable densities. Prevailing high temperatures in certain areas promote the drying of soils. This can be beneficial with soils of high plasticity but, generally, greater care is necessary to keep the moisture content of the soil as close as possible to the optimum for compaction with the particular compaction equipment in use.

Moisture contents well below the OMC (standard compaction) may be accepted, provided the compaction equipment and methods are adapted. In the arid areas of Ethiopia, this may reduce costs significantly. For silts and clays, the moisture content at the time of compaction should not exceed 105% of the OMC (standard compaction).

As indicated in Section 3.3, it is recommended that the upper 250 mm of soil immediately beneath the sub-base or capping layer, i.e. the top of the embankment fill or the natural subgrade, be compacted to a minimum of 100% of the maximum dry density obtained by ASTM D 698 (standard compaction). Alternatively, 93 % of the maximum dry density achieved by ASTM Test Method D 1557 (heavy compaction) may be specified. The same density should also be specified for fill behind abutments to bridges and for the backfill behind culverts. For the lower layers of an embankment, a compaction level of 90-93 per cent of the maximum dry density obtained by the heavy compaction is suitable, or a level of 95-100 per cent of the maximum density obtained by the light compaction.

During construction, compaction trials are to be carried out to determine the best way to achieve the specified density with the equipment available. Also during construction, it is not always easy to obtain an accurate measure of field density on site. The standard traditional methods of measurement are tedious, not particularly reproducible, and it is difficult to carry out sufficient tests to define a reliable density distribution. This problem can be alleviated to a great extent by making use of nuclear density and moisture gauges, since such devices are quicker to use and the results are more reproducible than traditional methods. However, the instruments will usually need calibration for use with the materials in question if accurate absolute densities are required. It may also be advisable to measure the moisture contents using traditional methods.

4.2.8 Slope Protection

Protection against erosion from runoff water, from rainfall and also from wind is required for the side slopes of the embankments. This is normally done by providing vegetation. The specific method (planting, seeding) of establishing the vegetation cover may be left unspecified, provided the Contractor is held to a maintenance period (normally one year). Hydro-seeding has advantages that should be utilized. Details of retaining the topsoil should be suggested in the contract documents, but incentives should be given to the Contractor to propose alternate methods. This favours the use of methods sanctioned by local experience.

4.3 Cuttings

Cuttings through sound rock can often stand at or near vertical, but in weathered rock or soil the conditions are more unstable. Instability is usually caused by an accumulation of water in the soil, and slips occur when this accumulation of water reduces the natural cohesion of the soil and increases its mass. Thus the design and construction of the road should always promote the rapid and safe movement of water from the area above the road to the area below, and under no circumstances should the road impede the flow of water or form a barrier to its movement.

4.3.1 Slope Stability

Methods of analysing slope stability are usually based on measurements of the density, moisture content and strength of the soil together with calculations of the stresses in the soil using classic slip-circle analyses. This type of analysis assumes that the soil mass is uniform. Sometimes failures do indeed follow the classic slip-circle pattern, but uniform conditions are rare, particularly in residual soils, and it is more common for slips to occur along planes of weakness in the vertical profile. Nevertheless, slope stability analysis remains an important tool in investigating the likely causes of slope failures and in determining remedial works, and such an analysis may be a necessary component of surveys to help design cuttings in soils.

Additional considerations regarding slope stability in cut sections are given in the *ERA Site Investigation Manual.*

4.3.2 Surveys

The construction of cuttings invariably disturbs the natural stability of the ground by the removal of lateral support and a change in the natural ground water conditions. The degree of instability will depend on the dip and stratification of the soils relative to the road alignment, the angle of the slopes, the ground water regime, the type of material, the dimensions of the cut, and numerous other variables. A full investigation is therefore an expensive exercise but, fortunately, most cuttings are small and straightforward. Investigations for the most difficult situations are best left to specialists. Local experience is an invaluable tool and every opportunity should be taken to maintain a local database.

An important part of a survey is to examine the performance of both natural and man-made slopes in the soils encountered along the length of the road, to identify the existing forms of failures, and to make the best possible use of the empirical evidence available in the area.

Where well defined strata appear in the parent rock, it is best to locate the road over ground where the layers dip towards the hill and to avoid locating the road across hillsides where the strata are inclined in the same direction as the ground surface.

During the survey, all watercourses crossing the road line must be identified and the need for culverts and erosion control established.

4.3.3 Design and Construction

The angle of cutting faces will normally be defined at the survey stage. Benching of the cut faces can be a useful construction expedient enabling the cutting to be excavated in well-defined stages and simplifying access for subsequent maintenance. The slope of the inclined face cannot usually be increased when benching is used and therefore the volume of earthworks is increased substantially. The bench itself can be inclined either outwards to shed water down the face of the cutting or towards the inside. In the former, surface erosion may pose a problem. In the latter, a paved drain will be necessary to prevent the concentration of surface water causing instability in the cutting.

A similar problem applies to the use of cut-off drains at the top of the cutting, which are designed to prevent runoff water from the area above the cutting from adding to the run-off problems on the cut slope itself. Unless such drains are lined and properly maintained to prevent water from entering the slope, they can be a source of weakness.

Control of ground water in the cut slopes is sometimes necessary. Various methods are available but most are expensive and complex, and need to be designed with care. It is advisable to carry out a proper ground water survey to investigate the quantity and location of sources of water.

As with embankments, it is essential that provision is made to disperse surface water from the formation at all stages of construction. Subsoil drains at the toe of the side slopes may be necessary.

The subsequent performance, stability and maintenance of cuttings will depend on the measures introduced to alleviate the problems created by rainfall and ground water. It is much more cost effective to install all the necessary elements at construction stage rather than to rely on remedial treatment later.

5 DRAINAGE AND SHOULDERS

5.1 Introduction

Road drainage design is the general term that is applied to two separate topics namely internal and external drainage.

5.1.1 Internal road drainage

The process of minimising the quantity of water that remains within a road pavement. This is done by maximising the ability of the road to lose water to an external drainage system. Sometimes this definition also includes minimising the quantity of water that gets into a road pavement in the first place.

5.1.2 External drainage

This consists of three components:

a) The process of determining the quantity of water that falls upon the road itself and its associated works that needs to be channelled away from the road by the drainage system. This is water that falls upon the road as rain.

b) The process of determining the quantity of water that flows in the streams, rivers and natural drains that the road has to cross. This is water that falls as rainfall at locations away from the road.

c) Design of the individual engineering features of the drainage system to accommodate the flow of water.

External drainage is the subject of a separate manual namely the ERA *Drainage Design Manual.* This manual is concerned with the structural design of road pavements and therefore deals with the provisions that need to be made for drainage within the pavement structure.

5.2 Sources of moisture entry into a pavement

Moisture is the single most important factor affecting pavement performance and long-term maintenance costs. Thus one of the main challenges faced by the designer is to provide a pavement structure in which the detrimental effects of moisture are contained to acceptable limits in relation to the traffic loading, nature of the materials being used, construction/maintenance provisions and degree of acceptable risk.

The various sources (and causes) of water ingress to, and egress from, a pavement are listed in Table 5.1. The two moisture zones in the pavement which are of critical significance are the equilibrium zone and the zone of seasonal moisture variation (see Figure 5.1). In sealed pavements over a deep water table, moisture contents in the equilibrium zone normally reach an equilibrium value after about two years from construction and remain reasonably constant thereafter (see Chapter 3).

In the zone of seasonal variation, the pavement moisture does not reach equilibrium and fluctuates with variation in rainfall. Generally, this zone is wetter than the equilibrium zone in the rainy season and it is drier in the dry season. Thus, the edge of the pavement is of extreme importance to ultimate pavement performance, with or without paved shoulders, and *is the most failure-prone region of a pavement when moisture conditions are relatively severe.*

Table 5-1 Sources of water ingress to, and egress from a road pavement

Means of Water Ingress	Explanation
Through the pavement surface	Through cracks due to pavement failure
	Penetration through intact layers
From the subgrade	Artesian head in the subgrade
	Pumping action at formation level
	Capillary action in the sub-base
From the road margins	Seepage from higher ground, particularly in cuttings
	Reverse falls at formation level
	Lateral/median drain surcharging
	Capillary action in the sub-base
	Through an unsealed shoulder collecting pavement and ground run-off
Through hydrogenesis (the aerial well effect)	Condensation and collection of water from vapour phase onto underside of an impermeable surface
Means of Water Egress	**Explanation**
Through the pavement surface	Through cracks under pumping action through the intact surfacing
Into the subgrade	Soakaway action
	Subgrade suction
To the road margins	Into lateral/median drains under gravitational flow in the sub-base
	Into positive drains through cross-drains acting as collectors

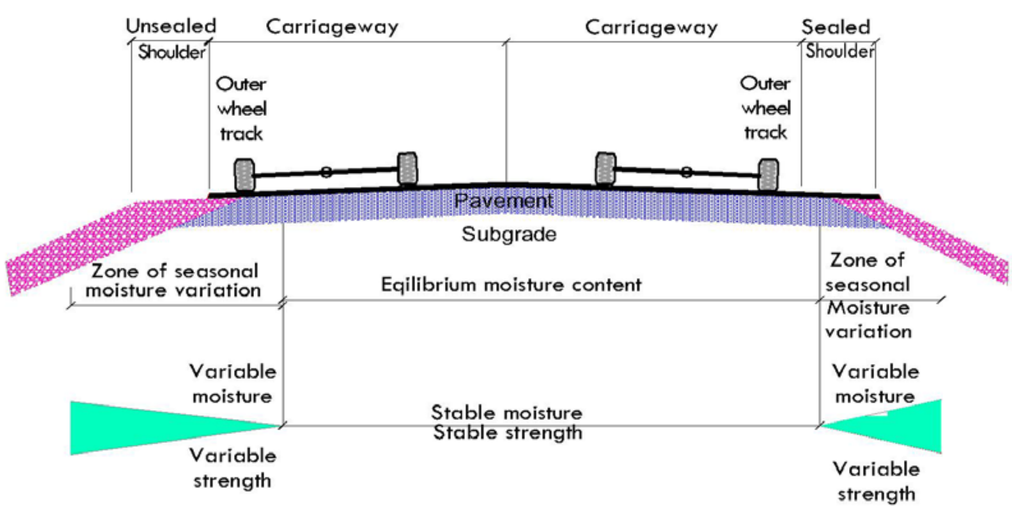


Figure 5-1 Moisture zones in a road pavement

5.3 Permeability

Permeability is a measure of the ease with which water passes through a material and is one of the key material parameters affecting drainage. Moisture ingress into and egress from a pavement is influenced by the permeability of the pavement, subgrade and surrounding materials. The relative permeability of adjacent materials may also govern moisture conditions. A significant decrease in permeability with depth or across boundaries between materials (i.e. permeability inversion) can lead to saturation of the materials in the vicinity of the inversion and should be avoided if there is a choice. Typical permeability values for saturated soils are shown in Table 5.2.

Table 5-2 Typical material permeability

Material	Permeability	Description
Gap-graded crushed rock	> 30 mm/s	Free draining
Gravel	> 10 mm/s	
Coarse sand	> 1 mm/s	
Medium sand	1 mm/s	Permeable
Fine sand	10 µm/s	
Sandy loam	1 µm/s	Practically impermeable
Silt	100 nm/s	
Clay	10 nm/s	Impermeable
Bituminous surfacing(1)	1 nm/s	

Note 1 Applies to well-maintained double chip seal. Thicker asphaltic concrete layers can exhibit significant permeability as a result of a linking of air voids. Permeability increases as the void content of the mix increases, with typical values ranging from 300µm/s at 2% air voids to 30µm/s at 12% air voids. Typically, a 1% increase in air voids content will result in a three-fold increase in permeability.

5.4 Achieving effective internal drainage

5.4.1 Side drainage and crown height above drain invert

Side drainage is one of the most significant factors affecting pavement performance (Figure 5.2) and can be quantified in terms of the 'drainage factor'. This is the product of the height of the crown of the road above the bottom of the ditch (h) and the horizontal distance from the centre-line of the road to the bottom of the ditch (d). This classification is shown in Table 5.3. A minimum value for h of 0.75m is recommended as illustrated below.

Irrespective of climatic region, if the site has effective side drains and adequate crown height, then the in situ subgrade strength should remain above the design value. If the drainage is poor, the in situ strengths will fall to below the design value.

Table 5-3 Classification of road drainage

Drainage Factor DF = d x h	Classification
< 2.5	Very poor
2.6 – 4.9	Poor
5.0 – 7.5	Moderate
> 7.5 or free draining	Good

Note: Classification can move up one class if longitudinal gradient is >1%

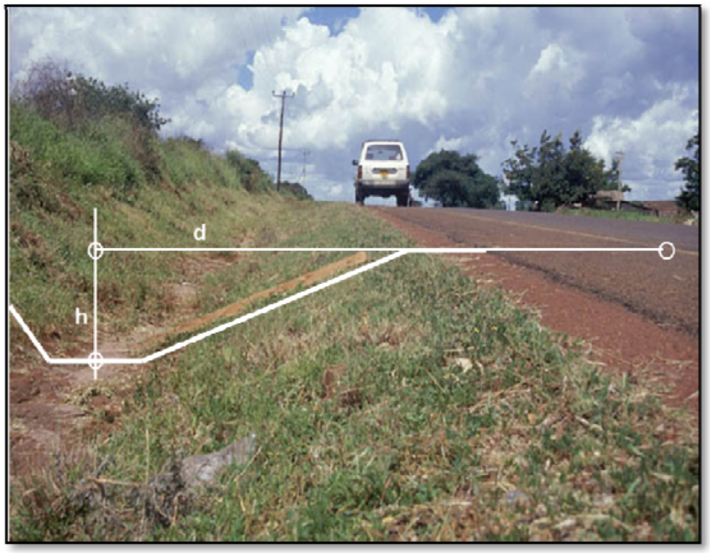


Figure 5-2 Example of a well-drained pavement

(drainage factor DF(d x h) >7.5)

5.4.2 Drainage within pavement layers

Drainage within the pavement layers themselves is an essential element of structural design because the strength of the subgrade in service depends critically on the moisture content during the most likely adverse conditions. Since it is impossible to guarantee that road surfaces will remain waterproof throughout their lives, it is critical to ensure that water is able to drain away quickly from within the pavement. This can be achieved by a number of measures as follows:

Permeability inversion

A permeability inversion exists when the permeability of the pavement and subgrade layers decreases with depth. If rain water enters the pavement from above, for example, through a poorly maintained shoulder, there is potential for moisture accumulation at the interface of the layers. The creation of such a perched water table could lead to saturation in the shoulder and lateral wetting beneath the running surface. This, in turn, may lead to base or sub-base saturation in the outer wheel track and result in failure of the base layer when trafficked. A permeability inversion often occurs at the interface between sub-base and subgrade since many subgrades are cohesive fine-grained materials.

It is therefore desirable for good internal drainage that a permeability inversion is avoided if this is possible with the materials available. It is achieved by ensuring that the permeability of the pavement and subgrade layers are at least equal or are increasing with depth. For example, the permeability of the base must be less than or equal to the permeability of the sub-base in a three-layered system. However, avoiding an inversion at subgrade level is often very difficult to achieve because many fine-grained subgrades are not very permeable. If this is the case, other design features which promote drainage become more important.

Where permeability inversion is unavoidable, the road shoulder should be sealed to an appropriate width to ensure that the lateral wetting front does not extend under the outer wheel track of the pavement and drainage across the pavement through the shoulders must be provided (Section 5.4). Indeed, these measures are advisable for removing water whether a permeability inversion exists or not.

In addition, lateral drainage can be encouraged by constructing the pavement layers with an exaggerated crossfall. This can be achieved by constructing the top of the sub-base with a crossfall of 3-4% and the top of the subgrade with a crossfall of 4-5%. Although this is not an efficient way to drain the pavement it is inexpensive and therefore worthwhile, particularly as full under-pavement drainage is rarely likely to be economically justified except for the highest road classes. [This also has the added benefit that the edge of the running surface i.e. the outer wheel track and the area most prone to failure, is slightly thicker than the centre of the road.]

An additional benefit is that the exaggerated crossfall at sub-base and subgrade level minimises the risk of creating a 'bird bath' effect. Such an effect occurs if a relatively impermeable subgrade or sub-base is deformed creating a dished shape from which entrapped water simply cannot drain laterally. The material simply remains wet and weak for an extended period and failure becomes very likely.

Figure 5.3 illustrates the recommended drainage arrangements

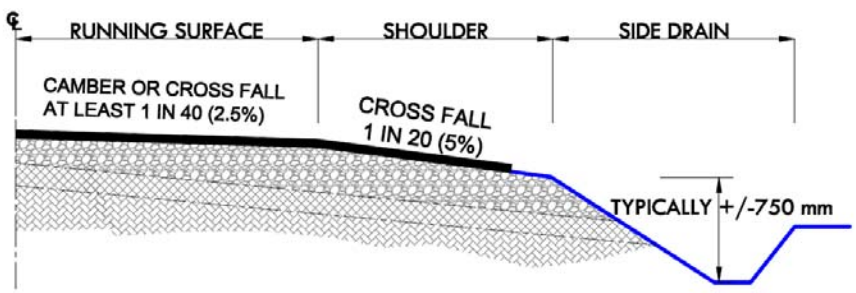


Figure 5-3 Recommended drainage arrangements

5.5 Shoulders

Shoulders participate in the structural function of a road pavement, providing lateral support for the pavement layers. They help in removing surface water from the road surface and facilitate the internal drainage of the pavement. They are especially important when unbound materials are used in the pavement.

The main materials to be considered for constructing the shoulders are:

i) The same materials as those used for the base and sub-base of the pavement (preferred alternative); or
ii) Gravel materials.

Cement or lime-treated materials may also be considered if they are used elsewhere in the pavement.

If gravel materials (unbound) are used for the construction of the shoulders, they should be of a quality similar to those described for sub-base or for gravel wearing courses (see Section 6.4)

For gravel roads the shoulders should be constructed with the same materials as the wearing course.

5.5.1 Ensuring proper shoulder design

When permeable roadbase materials are used, particular attention must be given to the drainage of this layer. Ideally, the roadbase and sub-base should extend right across the shoulders to the drainage ditches. In addition, proper crossfall on ***all*** layers is needed to assist the shedding of water into the side drains. A suitable value for paved roads is about 3% for the carriageway, with a slope of about 4-6% for the shoulders. [Increased crossfall is required for unsurfaced roads].

Lateral drainage can also be encouraged by constructing the pavement layers with an exaggerated crossfall, especially where a permeability inversion occurs. This can be achieved by constructing the top of the sub-base with a crossfall of 3-4% and the top of the subgrade with a crossfall of 4-5%. Although this is not an efficient way to drain the pavement it is inexpensive and therefore worthwhile, particularly as full under-pavement drainage is rarely likely to be economically justified except for the highest road classes. Figure 5.3 illustrates the recommended drainage arrangements.

If it is too costly to extend the roadbase and sub-base material across the shoulder, drainage channels at 3m to 5m intervals should be cut through the shoulder to a depth of 50mm below sub-base level. These channels should be back-filled with material of roadbase quality but which is more permeable than the roadbase itself, and should be given a fall of 1 in 10 to the side ditch. Alternatively, a preferable option would be to provide a continuous layer of pervious material of 75mm to 100mm thickness laid under the shoulder such that the bottom of the drainage layer is at the level of the top of the sub-base.

Generally, the drainage layer should be omitted on the upper side of super-elevated sections because this then provides a means of water entry.

When the base course can confidently be considered impervious, then the internal drainage is of lesser consequence. However, impervious materials should still be used for the shoulder, and it is preferable to provide them with surfacing.

5.5.2 Avoiding trench construction

Under no circumstances should the trench (or boxed-in) type of cross-section be used in which the pavement layers are confined between continuous impervious shoulders. This type of construction has the undesirable feature of trapping water at the pavement/shoulder interface and inhibiting flow into drainage ditches which, in turn, facilitates damage to the shoulders under even light traffic.

5.5.3 Unsealed shoulders

Unsurfaced shoulders generally require high maintenance and are not generally recommended. They must not be used if the layers below are pervious (e.g. extended pervious base course). They may be provided with topsoiling and seeding. If gravel shoulders are left unsurfaced, the extra width given to the base course (over the road surface width) should nevertheless be primed and sealed (see Section 5.5). This edge seal should also extend over the shoulder.

A common problem associated with the use of unsealed shoulders is water infiltration into the base and sub-base for the following reasons illustrated in Figure 5.4:

(a) Rutting adjacent to the sealed surface;
(b) Build-up of deposits of grass and debris;
(c) Poor joint between the base and shoulder (more common when a paved shoulder has been added after initial construction).

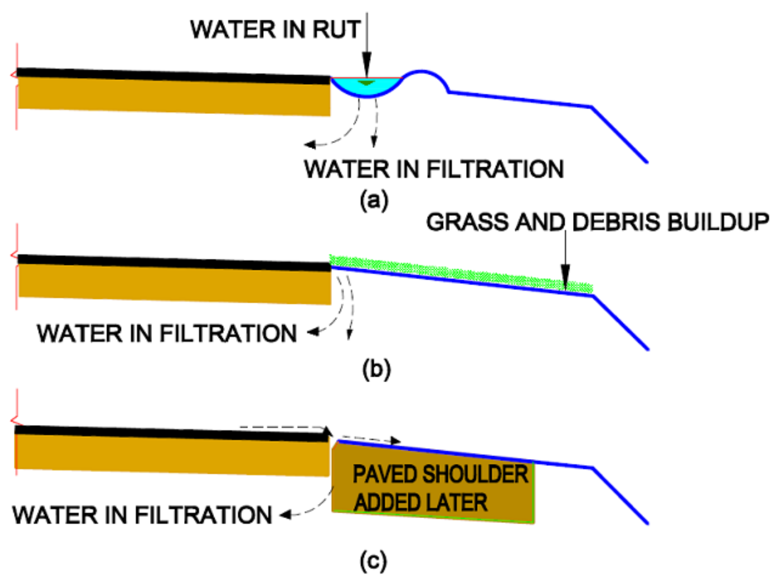


Figure 5-4 Typical drainage deficiencies associated with unsealed shoulders

5.5.4 Sealed shoulders

For a paved road it is usually economically justifiable to provide paved rather than unpaved shoulders. This is because it greatly improves pavement performance by ensuring that the zone of seasonal moisture variation does not penetrate to the outer wheel track (see Figure 5.1) and it:

i) Reduces erosion of the shoulders (especially on steep gradients).
ii) Reduces maintenance costs by avoiding the need for regravelling at regular intervals.
iii) Reduces the risk of road accidents, especially where the edge drop between the shoulder and the pavement is significant or the shoulders are relatively soft.

The shoulders should be sealed to a width of at least 1.0 from the edge of the sealed running surface, however, provision is often required for non-motorized traffic of various kinds and shoulder widths are variable (see ERA's *Geometric Design Manual)* hence the width of the sealed shoulder is also variable.

Paved or sealed shoulders should be differentiated from the carriageway e.g. by the use of edge markings.

5.6 Typical pavement cross-sections

Based on the above considerations, four alternative cross-sections are shown in Figure 5.5. It should be noted that, unless the base course material is extended fully across the shoulders, some extra width is nevertheless provided for the base. This provides support to the edge of the pavement, where compaction is difficult to achieve. The extra width of the base course should be about 30 cm. The edge seal covering the extra width of the base and the joint should extend a total of 40 to 60 cm.

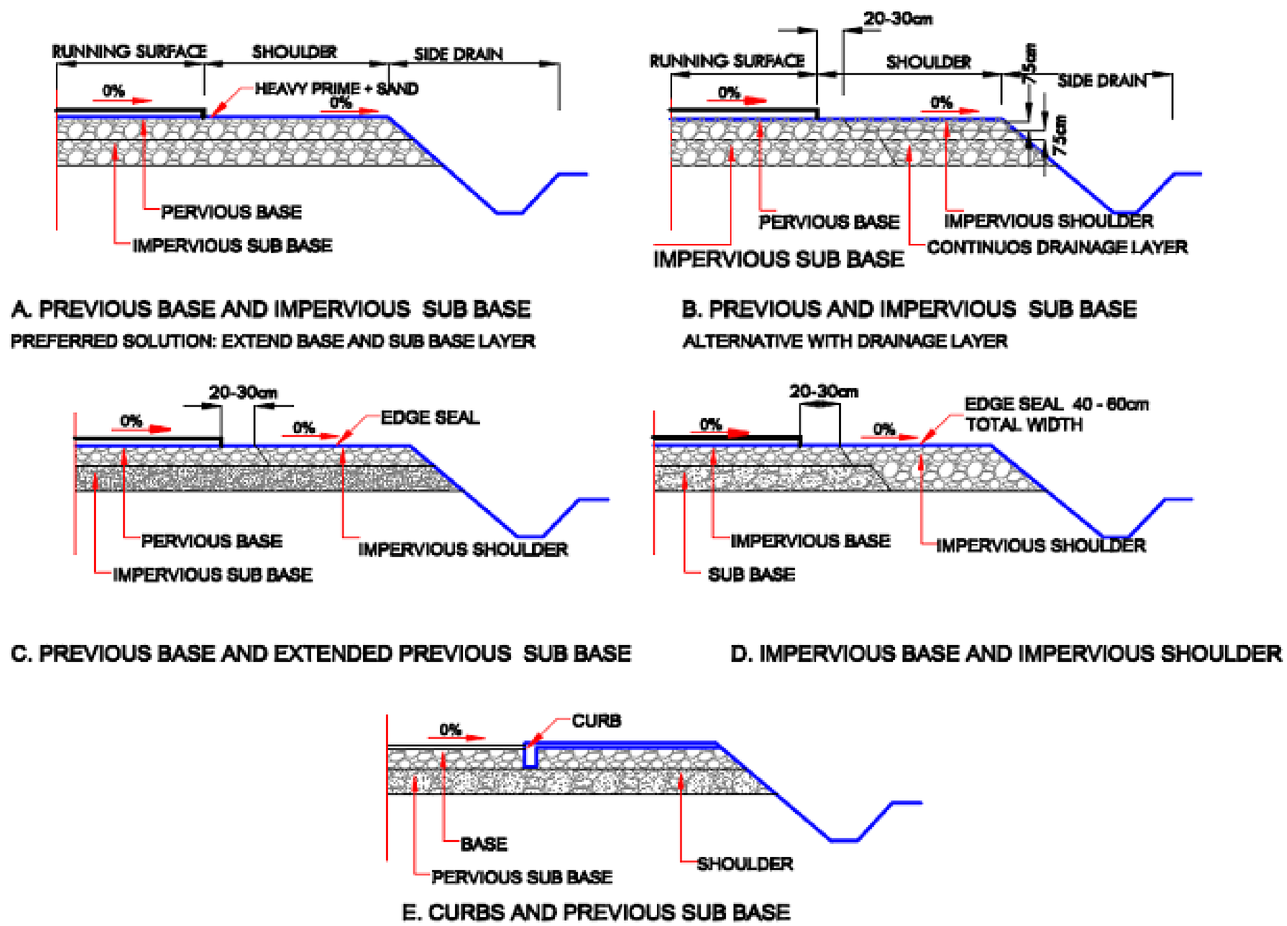
A fifth cross-section is also shown, using curbs, as is occasionally required in urban areas. It is to be noted that, since the drainage of the base course is impeded, it is essential that internal drainage be provided by a pervious sub-base or a drainage layer.

5.7 Adopting a holistic and integrated approach

The drainage measures highlighted above are all aimed at:

i) Preventing water from entering the pavement in the first place;
ii) Facilitating its outflow as quickly as is reasonable;
iii) Ensuring that the presence of water in the road for an extended period of time does not cause failures.

It should be appreciated, however, that the adoption of any single measure on its own is unlikely to be as effective as the adoption of a judicious mixture of a number of complementary measures applied simultaneously.



Note that for simplicity, changes in crossfall have not been shown.

Figure 5-5: Typical pavement cross sections illustrating the drainage arrangements

6 UNBOUND PAVEMENT MATERIALS

This chapter gives guidance on the selection of unbound materials for use as base course, sub-base, capping and selected subgrade layers. For lightly trafficked roads the requirements set out below may be too stringent and reference should be made to the *Low Volume Roads Design Manual.*

The main categories with a brief summary of their characteristics are shown in Table 6.1.

Table 6-1 Properties of Unbound Materials

Code	Description	Summary of Specification
GB1	Fresh, crushed rock	Dense graded, unweathered crushed stone, non-plastic parent fines
GB2	Crushed weathered rock, gravel or boulders	Dense grading, PI < 6, soil or parent fines; PP <60
GB2A	Dry-bound and Water-bound Macadam	Aggregate properties as for GB2 (see text) PI < 6: PP < 60
GB3	Natural coarsely graded granular material, including processed and modified gravels	Dense grading, PI < 6 CBR after soaking > 80%
GS	Natural gravel	CBR after soaking > 30%
GC	Gravel or gravel-soil	Dense graded; CBR after soaking > 15%

Notes: 1. These specifications are sometimes modified according to site conditions, material type and principal use (see text).
2 PP = Plasticity Product = PI x (per cent passing 0.075mm sieve)
2. GB = Granular base course, GS = Granular sub-base, GC = Granular capping layer.

6.1 Base Course Materials

A wide range of materials can be used as unbound base course including crushed quarried rock, crushed and screened, mechanically stabilised, modified or naturally occurring ‘as dug’ or ‘pit run’ gravels. Their suitability for use depends primarily on the design traffic level of the pavement and climate. However, all base course materials must have a particle size distribution and particle shape which provide high mechanical stability and should contain sufficient fines (amount of material passing the 0.425 mm sieve) to produce a dense material when compacted.

In circumstances where several suitable types of base course materials are available, the final choice should take into account the expected level of future maintenance and the total costs over the expected life of the pavement.

The use of locally available materials is encouraged, particularly at traffic levels of less than 0.7 million ESAs (i.e. categories T1 and T2, Table 2.5). Their use is described in detail in the *Low Volume Roads Design Manual.* It is based on the results of performance studies and incorporates special design features which ensure their satisfactory performance.

6.1.1 Crushed Stone

Graded crushed stone (GB1)

This material is produced by crushing fresh, quarried rock (GB1) and may be an all-in product, usually termed a 'crusher-run', or alternatively the material may be separated by screening and recombined to produce a desired particle size distribution, as per the specifications. Alternate gradation limits, depending on the local conditions for a particular project, are shown in Table 6.2. After crushing, the material should be angular in shape with a Flakiness Index (British Standard 812, Part 105) of less than 35%, and preferably of less than 30%. If the amount of fine aggregate produced during the crushing operation is insufficient, non-plastic angular sand may be used to make up the deficiency. In constructing a crushed stone base course, the aim should be to achieve maximum impermeability compatible with good compaction and high stability under traffic.

Table 6-2 Grading Limits for Graded Crushed Stone Base Course Materials (GB1)

Test sieve (mm)	Percentage by mass of total aggregate passing test sieve		
	Nominal maximum particle size		
	37.5 mm	28 mm	20 mm
50	100	-	-
37.5	95 – 100	100	-
28	-	-	100
20	60 – 80	70 - 85	90 – 100
10	40 – 60	50 - 65	60 – 75
5	25 - 40	35 - 55	40 – 60
2.36	15 – 30	25 - 40	30 – 45
0.425	7 – 19	12 - 24	13 – 27
0.075 (1)	5 – 12	5 - 12	5 – 12

Note 1. For paver-laid materials a lower fines content may be accepted.

To ensure that the materials are sufficiently durable, they should satisfy the criteria given in Table 6.3. These are a minimum Ten Per Cent Fines Value (TFV) (British Standard 812, Part 111) and limits on the maximum loss in strength following a period of 24 hours of soaking in water. The likely moisture conditions in the pavement are taken into account in broad terms based on annual rainfall. Alternatively, requirements expressed in terms of the results of the Aggregate Crushing Value (ACV) (British Standard 812, Part 110) may be used. The ACV should, preferably, be less than 25 and always less than 29. Other simpler tests e.g. the Aggregate Impact Test (British Standard 812, Part 112, 1990**)** may be used in quality control testing provided a relationship between the results of the chosen test and the TFV has been determined. Unique relationships do not exist between the results of the various tests but good correlations can be established for individual material types and these need to be determined locally.

Table 6-3 Mechanical Strength Requirements (Ten Per Cent Fines Test) for the Aggregate Fraction of Crushed Stone Base Course Materials (GB1)

Typical Annual Rainfall (mm)	Minimum Ten Percent Fines Values (kN)	Minimum Ratio Wet/Dry Test (%)
>500	110	75
<500	110	60

When dealing with materials originating from the weathering of basic igneous rocks, the recommendations in Section 6.1.3 should be used.

The fine fraction of a GB1 material should be non-plastic.

These materials may be dumped and spread by grader but it is preferable to use a paver to ensure that the completed surface is smooth with a tight finish. The material is usually kept wet during transport and laying to reduce the likelihood of particle segregation. Thus they are often called ‘wet mix’ and should not be confused with ‘water-bound macadam’ (see below).

The in situ dry density of the placed material should be a minimum of 98% of the maximum dry density obtained in the ASTM Test Method D 1557 (Heavy Compaction). The compacted thickness of each layer should not exceed 200 mm.

Crushed stone base courses constructed with proper care with the materials described above should have CBR values well in excess of 100 per cent. There is usually no need to carry out CBR tests during construction.

***Dry-bound and Water-bound Macadam (GB2A*)**

Dry-bound macadam is a traditional form of construction and its performance can be comparable to that of a graded crushed stone. It is particularly applicable in areas where water is scarce or expensive to obtain and it is also suitable where labour intensive construction is an economic option. The materials consist of nominal single-sized crushed stone and non-plastic fine aggregate (passing the 5.0 mm sieve). The fine material should preferably be well graded and consist of crushed rock fines or natural, angular pit sand.

The dry-bound macadam process involves laying single-sized crushed stone of either 37.5 mm or 50 mm nominal size in a series of layers to achieve the design thickness. The compacted thickness of each layer should not exceed twice the nominal stone size. Each layer of coarse aggregate should be shaped and compacted and then the fine aggregate spread onto the surface and vibrated into the interstices to produce a dense layer. Any loose material remaining is brushed off and final compaction carried out, usually with a heavy smooth-wheeled roller. This sequence is then repeated until the design thickness is achieved. To aid the entry of the fines, the grading of the 37.5 mm nominal size stone should be towards the coarse end of the recommended range. Economy in the production process can be obtained if layers consisting of 50 mm nominal size stone and layers of 37.5 mm nominal size stone are both used to allow the required total thickness to be obtained more precisely and to make better overall use of the output from the crushing plant.

Water-bound macadam is similar to dry-bound macadam. It also consists of two components namely a relatively single-sized stone with a nominal maximum particle size of 50 mm or 37.5 mm and well graded fine aggregate which passes the 5.0 mm sieve. The coarse material is usually produced from quarrying fresh rock. The crushed stone is laid, shaped and compacted and then fines are added, rolled and *washed* into the surface to produce a dense material. Care is necessary in this operation to ensure that water sensitive plastic materials in the sub-base or subgrade do not become saturated. The compacted thickness of each layer should not exceed twice the maximum size of the stone. The fine material should preferably be non-plastic and consist of crushed rock fines or natural, angular pit sand.

Typical grading limits for the coarse fraction of GB2A materials are given in Table 6.4. The grading of M2 and M4 correspond with nominal 50 mm and 37.5 mm single-sized aggregates (British Standard 63 (1987)) and are appropriate for use with mechanically crushed aggregate. M1 and M3 are broader specifications. M1 has been used for hand-broken stone but if suitable screens are available, M2, M3 and M4 are preferred.

Aggregate hardness, durability, particle shape and in situ density should each conform to those given above for graded crushed stone.

Table 6-4 Coarse Aggregate Gradings for Dry-bound and Water-bound Macadam

Test sieve size (mm)	Percentage by mass of total aggregate passing test sieve			
	M1	**M2**	**M3**	**M4(1)**
75	100	100	100	
50	85 - 100	85 - 100	85 - 100	100
37.5	35 - 70	0 - 30	0 - 50	85 - 100
28	0 - 15	0 - 5	0 - 10	0 - 40
20	0 - 10			0 - 5

Note 1 To aid entry of the fines, the coarser end of this grading is preferred.

6.1.2 Naturally Occurring Granular Materials, Boulders, Weathered Rocks

Normal requirements for natural gravels and weathered rocks (GB2, GB3)

A wide range of materials including lateritic, calcareous and quartzitic gravels, river gravels, boulders and other transported gravels, or granular materials resulting from the weathering of rocks can be used successfully as base course materials. Table 6.5 contains three recommended particle size distributions for suitable materials corresponding to maximum nominal sizes of 37.5 mm, 20 mm and 10 mm. Only the two larger sizes should be considered for traffic in excess of 1.5 million ESAs. To ensure that the material has maximum mechanical stability, the particle size distribution should be approximately parallel with the grading envelope.

To meet the requirements consistently, screening and crushing of the larger sizes may be required. The fraction coarser than 10 mm should consist of more than 40 per cent of particles with angular, irregular or crushed faces. The mixing of materials from different sources may be warranted in order to achieve the required grading and surface finish. This may involve adding fine or coarse materials or combinations of the two.

Table 6-5 Recommended Particle Size Distributions for Mechanically Stable Natural Gravels & Weathered Rocks for use as Base Course Material (GB2, GB3)

Test sieve (mm)	Percentage by mass of total aggregate passing test sieve		
	Nominal maximum particle size		
	37.5 mm	20 mm	10 mm
50	100	-	-
37.5	80 – 100	100	-
20	60 – 80	80 – 100	100
10	45 – 65	55 – 80	80 – 100
5	30 – 50	40 – 60	50 – 70
2.36	20 – 40	30 – 50	35 – 50
0.425	10 – 25	12 – 27	12 – 30
0.075	5 – 15	5 – 15	5 – 15

All grading analyses should be done on materials that have been compacted. This is especially important if the aggregate fraction is susceptible to breakdown under compaction and in service. For materials whose stability decreases with breakdown, an aggregate hardness based on a minimum soaked Ten Per Cent Fines Value of 50 kN may be specified.

The fines of these materials should preferably be non-plastic but should normally never exceed a PI of 6.

If the PI approaches the upper limit of 6, it is desirable that the fines content be restricted to the lower end of the range. To ensure this, a maximum PP of 60 is recommended or alternatively a maximum Plasticity Modulus (PM) of 90 where:

PM = PI x (percentage passing the 0.425 mm sieve)

If difficulties are encountered in meeting the plasticity criteria, consideration should be given to modifying the material by the addition of a low percentage of hydrated lime or cement.

When used as a base course, the material should be compacted to a density equal to or greater than 98 per cent of the maximum dry density achieved in the ASTM Test Method D 1557 (Heavy Compaction). When compacted to this density in the laboratory, the material should have a minimum CBR of 80% after four days immersion in water (ASTM D 1883).

Low volume roads

For low volume roads (Chart A and traffic classes T1 and T2) the plasticity and strength requirements for the unbound materials can be relaxed, especially when the subgrade is strong and/or the climate is dry. In Ethiopia, the low altitude areas of the northeast (low areas of Tigray, Welo and Hererge regions) and southeast (Hererge and Bale) are dry throughout most of the year. In these low rainfall areas, typically with a mean annual rainfall of less than 500 mm, and where evaporation is high, moisture conditions beneath a well sealed surface are unlikely to rise above the optimum moisture content determined in the ASTM Test Method D 1557 (Heavy Compaction). In such conditions, high strengths (CBR>80 %) are likely to develop even when natural gravels containing a substantial amount of plastic fines are used. In these situations and depending on subgrade strength, for the lowest traffic categories the maximum allowable PI can be increased to 9 and the minimum soaked CBR criterion reduced to 65% at the expected field density (for full details see the *Low Volume Roads Manual*).

Materials of basic igneous origin

Materials in this group are sometimes weathered and may release additional plastic fines during construction or in service. Problems are likely to worsen if water enters the pavement and this can lead to rapid and premature failure. The state of decomposition also affects their long-term durability when stabilised with lime or cement. The group includes common rocks such as basalts and dolerites but also covers a wider variety of rocks and granular materials derived from their weathering, transportation or other alteration.

Normal aggregate tests are often unable to identify unsuitable materials in this group. Even large, apparently sound particles may contain minerals that are decomposed and potentially expansive. The release of these minerals may lead to a consequent loss in bearing capacity. There are several methods of identifying unsound aggregates. These include petrographic analysis to detect secondary (clay) minerals and the use of various chemical soundness tests, e.g. sodium or magnesium sulphate (ASTM C 88). Indicative limits based on these tests include:

(a) A maximum secondary mineral content of 20%,

(b) A maximum loss of 12% or 20% after 5 cycles in the sodium or magnesium sulphate tests respectively.

In most cases it is advisable to seek expert advice when considering their use, especially when new deposits are being evaluated. It is also important to subject the material to a range of tests since no specific method can consistently identify problem materials. Recommendations for appropriate test and limits for the durability of road basecourse materials can be found in Sampson (1991).

In some areas of Ethiopia, weathered basalt gravels are available in large quantities. To study the performance of weathered basalt gravel, experimental roads were constructed in Ethiopia, namely on the Gelenso-Mechara project and Ghion-Jimma project under a Joint Road Research Project of the Ethiopian Transport Construction Authority and TRRL (Beaven et al. 1988). Results indicated that these materials stabilised with 3 per cent of lime and surface dressed should provide an acceptable alternative to crushed stone base construction for main roads in Ethiopia. A particular advantage of this material is that it avoids the problem of clay working up into the base, which is a frequent source of failure when using crushed stone over active clay.

Materials of marginal quality.

Naturally occurring gravels which do not meet the normal specifications for base course materials have been used successfully. They include lateritic, calcareous and volcanic gravels. In general their use should be confined to the lower traffic categories (i.e. T1 and T2) unless local studies have shown that they have performed successfully at higher levels (for full details see the ERA *Low Volume Roads Design Manual)*.

Laterite gravels with plasticity index in the range of 6-12 and plasticity modulus in the range of 150-250 are recommended (CIRIA, 1995) for use as base course material for T3 level of traffic volume. The values towards the higher range are valid for semi-arid and arid areas of Ethiopia, i.e. with annual rainfall less than 500 mm.

The calcareous gravels, which include calcretes and marly limestones, deserve special mention. Typically, the plasticity requirements for these materials, all other things being equal, can be increased by up to 50% above the normal requirements in the same climatic area without any detrimental effect on the performance of otherwise mechanically stable bases. Strict control of grading is also less important and deviation from a continuous grading is tolerable.

Cinder gravels can also be used as a base course materials in lightly trafficked (T1 and T2) Chart A and Chart B roads (Newill et al. 1987).

6.2 Sub-Bases (GS)

The sub-base is an important load spreading layer in the completed pavement. It enables traffic stresses to be reduced to acceptable levels in the subgrade, it acts as a working platform for the construction of the upper pavement layers and it acts as a separation layer between subgrade and base course. Under special circumstances, it may also act as a filter or as a drainage layer. In wet climatic conditions, the most stringent requirements are dictated by the need to support construction traffic and paving equipment. In these circumstances, the sub-base material needs to be more tightly specified. In dry climatic conditions, in areas of good drainage, and where the road surface remains well sealed, unsaturated moisture conditions prevail and sub-base specifications may be relaxed. The selection of sub-base materials will therefore depend on the design function of the layer and the anticipated moisture regime, both in service and at construction.

6.2.1 Bearing Capacity

A minimum CBR of 30 per cent is required at the highest anticipated moisture content when compacted to the specified field density, usually a minimum of 95 per cent of the maximum dry density achieved in the ASTM Test Method D 1557 (Heavy Compaction). Under conditions of good drainage and when the water table is not near the ground surface (see Chapter 3) the field moisture content under a sealed pavement will be equal to or less than the optimum moisture content in the ASTM Test Method D 698 (Light Compaction). In such conditions, the sub-base material should be tested in the laboratory in an unsaturated state. Except in arid areas (Category 3 in Chapter 3), if the base course allows water to drain into the lower layers, as may occur with unsealed shoulders and under conditions of poor surface maintenance where the base course is pervious (see Section 3.1), saturation of the sub-base is likely. In these circumstances, the bearing capacity should be determined on samples soaked in water for a period of four days. The test should be conducted on samples prepared at the density and moisture content likely to be achieved in the field. In order to achieve the required bearing capacity, and for uniform support to be provided to the upper pavement, limits on soil plasticity and particle size distribution may be required. Materials which meet the recommendations of Tables 6.5 and 6.6 will usually be found to have adequate bearing capacity.

6.2.2 Use as a Construction Platform

In many circumstances the requirements of a sub-base are governed by its ability to support construction traffic without excessive deformation or ravelling. A high quality sub-base is therefore required where loading or climatic conditions during construction are severe. Suitable material should possess properties similar to those of a good surfacing material for unpaved roads. The material should be well graded and have a plasticity index at the lower end of the appropriate range for an ideal unpaved road wearing course under the prevailing climatic conditions. These considerations form the basis of the criteria given in Tables 6.6 and 6.7. Material meeting the requirements for severe conditions will usually be of higher quality than the standard sub-base (GS). If materials to these requirements are unavailable, trafficking trials should be conducted to determine the performance of alternative materials under typical site conditions.

Table 6-6 Recommended Plasticity Characteristics for Granular Sub-Bases (GS)

Climate	Typical Annual Rainfall (mm)	Liquid Limit	Plasticity Index	Linear Shrinkage
Moist tropical and wet tropical	> 500	< 35	< 6	< 3
Seasonally wet tropical	> 500	< 45	< 12	< 6
Arid and semi-arid	< 500	< 55	< 20	< 10

Table 6-7 Typical Particle Size Distribution for Sub-bases (GS)

Test Sieve (mm)	Percentage by mass of total aggregate passing test sieve (%)
50	100
37.5	80 – 100
20	60 – 100
5	30 – 100
1.18	17 – 75
0.3	9 – 50
0.075	5 – 25

In the construction of low-volume roads local experience is often invaluable and a wider range of materials may often be found to be acceptable.

In Ethiopia, laterite is one of the widely available materials and can be used as a sub-base material. Laterite meeting the gradation requirements of Table 6-7 can be used for traffic levels up to 3 million ESA provided the following criteria are satisfied:

i) CBR (%) (after soaking) > 30
ii) Plasticity Index (%) < 25
iii) Plasticity Modulus (PM) < 500

6.2.3 Sub-Base as a Filter or Separating Layer

This may be required to protect a drainage layer from blockage by a finer material or to prevent migration of fines and the mixing of two layers. The two functions are similar except that for use as a filter the material needs to be capable of allowing drainage to take place and therefore the amount of material passing the 0.075 mm sieve must be restricted.

The following criteria should be used to evaluate a sub-base as a separating or filter layer:

a) The ratio $\frac{\text{D15 (coarse layer)}}{\text{D85 (fine layer)}}$ should be less than 5

Where D15 is the sieve size through which 15% by weight of the material passes and D85 is the sieve size through which 85% passes.

b) The ratio $\frac{\text{D50 (coarse layer)}}{\text{D50 (fine layer)}}$ should be less than 25

For a filter to possess the required drainage characteristics a further requirement is:

c) The ratio $\frac{\text{D15 (coarse layer)}}{\text{D15 (fine layer)}}$ should lie between 5 and 40

These criteria may be applied to the materials at both the base course/sub-base and the sub-base/subgrade interfaces.

6.3 Selected Subgrade Materials and Capping Layers (GC)

These materials are often required to provide sufficient cover on weak subgrades. They are used in the lower pavement layers as a substitute for a thick sub-base to reduce costs, and a cost comparison should be conducted to assess their cost effectiveness.

In some of the design charts, substitution of part of the sub-base with GC quality material is allowed as mentioned in the footnotes to the charts. The substitution ratio is 1.3:1 so that 50mm of sub-base can be replaced with 65mm of GC, for example, provided that the rules in the footnotes are followed. Similarly, a layer of GC material on top of a weak subgrade effectively increases the subgrade class as illustrated in the design charts.

The requirements are less strict than for sub-bases. A minimum CBR of 15 per cent is specified at the highest anticipated moisture content measured on samples compacted in the laboratory at the specified field density. This density is usually specified as a minimum of 95 per cent of the maximum dry density in the ASTM Test Method D 1557 (Heavy Compaction). In estimating the likely soil moisture conditions, the designer should take into account the functions of the overlying sub-base layer and its expected moisture condition and the moisture conditions in the subgrade. If either of these layers is likely to be saturated during the life of the road, then the selected layer should also be assessed in this state. Recommended gradings or plasticity criteria are not given for these materials. However, it is desirable to select reasonably homogeneous materials since overall pavement behaviour is often enhanced by this. The selection of materials which show the least change in bearing capacity from dry to wet is also beneficial.

7 CEMENT AND LIME-STABILISED MATERIALS

7.1 Introduction

This chapter gives guidance on the manufacture and use of cement and lime-stabilised materials in base course, sub-base, capping and selected fill layers of pavements. The stabilising process involves the addition of a stabilising agent to the soil, mixing with sufficient water to achieve the optimum moisture content, compaction of the mixture, and final curing to ensure that the strength potential is realized.

Many natural materials can be stabilised to make them suitable for road pavements but this process is only economical when the cost of overcoming a deficiency in one material is less than the cost of importing another material which is satisfactory without stabilisation.

The primary use for cement and lime stabilisation in tropical countries like Ethiopia has so far been with gravelly soils to produce roadbases. The processes can also be used with more clayey soils to make the upper layer of sub-bases.

Stabilisation can enhance the properties of road materials and pavement layers in the following ways:

i) A substantial proportion of their strength is retained when they become saturated with water.

Surface deflections are reduced.

ii) Materials in the supporting layer cannot contaminate the stabilised layer.
iii) Layers above a stabilised layer can be compacted more effectively and thereby possess enhanced strength and elastic properties.
iv) Resistance to erosion is increased.
v) Lime-stabilised material is suitable for use as a capping layer or working platform when the in situ material is excessively wet or weak and removal is not economical.

Associated with these desirable qualities are several possible problems:

i) Traffic, thermal and shrinkage stresses can cause stabilised layers to crack.
ii) Cracks can reflect through the surfacing and allow water to enter the pavement structure.

If carbon dioxide has access to the material, the stabilisation reactions may be reversible and the strength of the layers can decrease.

iii) The construction operations require more skill and control than for the equivalent unstabilized material.

Methods of dealing with these problems are outlined in Section 7.6.

The minimum acceptable strength of a stabilised material depends on its position in the pavement structure and the level of traffic. It must be sufficiently strong to resist traffic stresses but upper limits of strength are usually set to minimize the risk of reflection cracking. Three types of stabilised layer have been used in the structural design catalogue (Chapter 10) and the strengths required for each are defined in Table 7.1.

Table 7-1 Properties of Cement and Lime-Stabilised Materials

Code	Description	Cement-stabilised	Lime stabilised
		Unconfined compressive strength* (MPa)	Minimum CBR value* (%)
CB1	Stabilised road base	3.0 - 6.0	100
CB2	Stabilised road base	1.5 - 3.0	80
CS	Stabilised sub-base	0.75 - 1.5	40

* Strength tests on 150 mm cubes (see Section 7.4)

7.2 The Stabilisation Process

When lime is added to a cohesive soil, calcium ions replace sodium ions in the clay fraction until the soil becomes saturated with calcium and the pH rises to a value in excess of 12 (i.e. highly alkaline). The quantity of lime required to satisfy these reactions is determined by the initial consumption of lime test (ICL), (British Standard 1924).

The solubility of silica and alumina in the soil increases dramatically when the pH is greater than 12 and their reaction with lime can then proceed, producing cementitious calcium silicates and aluminates. Amorphous silica reacts particularly well with lime. The cementitious compounds form a skeleton that holds the soil particles and aggregates together.

The primary hydration of cement forms calcium silicate and aluminate hydrates, releasing lime, which reacts with soil components, as described above, to produce additional cementitious material.

The gain in strength associated with the formation of calcium silicates and aluminates occurs slowly. It is accelerated by heat, an advantage when using lime stabilisation in hot climates.

7.3 Selection of Type of Treatment

The selection of the stabiliser is based on the plasticity and particle size distribution of the material to be treated. The appropriate stabiliser can be selected according to the criteria shown in Table 7.2.

Some control over the particle size distribution can be achieved by limiting the coefficient of uniformity to a minimum value of 5. The coefficient of uniformity is defined as the ratio of the sieve size through which 60 per cent of the material passes to the sieve size through which 10 per cent passes (D60/D10 in the nomenclature of Section 6.2.3). If the coefficient of uniformity lies below this value the cost of stabilisation will be high and the maintenance of cracks in the finished road could be expensive.

Except for materials containing amorphous silica, e.g. some sandstones and chert, material with low plasticity is usually best treated with cement. However, reactive silica in the form of pozzolans can be added to soils with low plasticity to make them suitable for stabilisation with lime. If the plasticity of the soil is high there are usually sufficient reactive clay minerals which can be readily stabilised with lime. Cement is more difficult to mix intimately with plastic materials but this problem can be alleviated by pre-treating the soil with approximately 2 per cent of lime to make it more workable. When lime is added to a plastic material, it flocculates the clay and substantially reduces the plasticity index.

Table 7-2 Guide to the Type of Stabilisation Likely to be Effective

<table>
<tr><th rowspan="3">Type of stabilisation</th><th colspan="6">Soil properties</th></tr>
<tr><th colspan="3">More than 25% passing the 0.075 mm sieve</th><th colspan="3">Less than 25% passing the 0.075 mm sieve</th></tr>
<tr><th>PI ≤l0</th><th>10<PI ≤20</th><th>PI>20</th><th>PI ≤6</th><th>PI ≤10</th><th>PI > 10</th></tr>
<tr><td>Cement</td><td>Yes</td><td>Yes</td><td>Note 1</td><td>Yes</td><td>Yes</td><td>Yes</td></tr>
<tr><td>Lime</td><td>Note 1</td><td>Yes</td><td>Yes</td><td>No</td><td>Note 1</td><td>Yes</td></tr>
<tr><td>Lime-Pozzolan</td><td>Yes</td><td>Note 1</td><td>No</td><td>Yes</td><td>Yes</td><td>Note 1</td></tr>
</table>

Notes. 1. The agent will have only a marginal effective
2. PP = Plasticity Product (see Chapter 6).

If possible, the quality of the material to be stabilised should meet the minimum standards set out in Table 7.3. Stabilised layers constructed from these materials are more likely to perform satisfactorily even if they are affected by carbonation during their lifetime. Materials not complying with Table 7.3 can sometimes be stabilised but more additive will be required and the cost and the risk from cracking and carbonation will increase.

Table 7-3 Desirable Properties of Material before Stabilisation

Test sieve (mm)	Percentage by mass of total aggregate passing sieve (mm)		
	CB1	**CB2**	**CS**
53	100	100	-
37.5	85 – 100	80 – 100	-
20	60 – 90	55 – 90	-
5	30 – 65	25 – 65	-
2	20 – 50	15 – 50	-
0.425	10 – 30	10 – 30	-
0.075	5 - 15	5 - 15	-
	Maximum allowable value		
LL	25	30	-
PI	6	10	20
LS	3	5	-

Note 1 It is recommended that materials should have a coefficient of uniformity of 5 or more.

Some aspects of construction must also be considered in selecting the stabiliser. It is not always possible to divert traffic during construction and the work must then be carried out in half-widths. The rate of gain of strength in the pavement layer may sometimes need to be rapid so that traffic can be routed over the completed pavement as soon as possible. Under these circumstances, cement stabilisation, with a faster curing period, is likely to be more suitable than lime stabilisation.

Certain types of organic compounds in soils can affect the hydration of cement and inhibit the gain in strength. It is recommended that the effects of organic matter are assessed by strength tests as outlined below.

Soils in which sulphates are present should be avoided. Examples have been reported of lime-stabilised clays swelling to a marked degree in the months following construction. The cause of this swelling has been traced to a reaction between sulphates in the soil and the calcium silica-alumina hydrates formed as the lime reacts with soil. This reaction can occur in the presence of as little as 0.3 per cent of sulphate in the soil and is reported to be activated in situations where the soil is in or near a saturated condition.

7.4 Cement Stabilisation

7.4.1 Selection of Cement Content

The cement content determines whether the characteristics of the mixture are dominated by the properties of the original soil or by the hydration products. As the proportion of cement in the mixture increases, so the strength increases. Strength also increases with time. During the first one or two days after construction this increase is rapid. Thereafter, the rate slows down although strength gain continues provided the layer is well cured. The choice of cement content depends on the strength required, the durability of the mixture, and the soundness of the aggregate.

The minimum cement content, expressed as a percentage of the dry weight of soil, should exceed the quantity consumed in the initial ion exchange reactions. It is recommended that the percentage of cement added should be equal to or greater than the ICL.

Additional stabiliser is normally incorporated to take account of the variability in mixing which occurs on site. If good control is exercised over the construction operations, an extra one per cent of stabiliser is usually satisfactory for this purpose.

7.4.2 Preparation of Specimens

The optimum moisture content and the maximum dry density for mixtures of soil plus stabiliser are determined according to British Standard 1924 for additions of 2, 4, 6 and 8 per cent of cement.

Samples for the strength tests should also be mixed and left for two hours (to account for delays in practice) before being compacted into 150 mm cubes at 97 per cent of the maximum dry density obtained, after a similar two hour delay, in the ASTM Test Method D 1557 (Heavy Compaction). These samples are then moist cured for 7 days and soaked for 7 days in accordance with BS 1924.

Two methods of moist curing are described in the Standard. The preferred method is to seal the specimens in wax but if this is not possible they must be wrapped in cling film and sealed in plastic bags. The specimens should be maintained at 25°C during the whole curing and soaking period.

When the soaking phase is completed, the samples are crushed, their strengths measured, and an estimate made of the cement content needed to achieve the target strength.

As an alternative, the strength of stabilised sub-base material may be measured by the CBR test after 7 days of moist curing and 7 days of soaking. A minimum CBR of 70 is recommended.

7.5 Lime Stabilisation

7.5.1 Properties of Lime-Stabilised Materials

When lime is added to a plastic material, it first flocculates the clay and substantially reduces the plasticity index. Both the ion exchange reaction and the production of cementitious materials increase the stability and reduce the volume change within the clay fraction. It is not unusual for the swell to be reduced from 7 or 8 per cent to 0.1 per cent by the addition of lime.

The ion exchange reaction occurs quickly and can increase the CBR of clayey materials by a factor of two or three. The reduction of plasticity is time dependent during the initial weeks, and has the effect of increasing the optimum moisture content and decreasing the maximum dry density in compaction. The compaction characteristics are therefore constantly changing with time and delays in compaction cause reductions in density and consequential reductions in strength and durability.

The workability of the soil also improves as the soil becomes more friable. If the amount of lime added exceeds the ICL, the stabilised material will generally be non-plastic or only slightly so.

The production of cementitious materials can continue for ten years or more but the strength developed will be influenced by the materials and the environment. The elastic modulus behaves similarly to the strength and continues to increase for a number of years. Between the ages of one month and two to three years there can be a four-fold increase in the elastic modulus.

7.5.2 Types of lime

The most common form of commercial lime used in lime stabilisation is hydrated high calcium lime, $Ca(OH)_2$, but monohydrated dolomitic lime, $Ca(OH)_2$, MgO, calcitic quick lime, CaO, and dolomitic quicklime, CaO, MgO are also used.

For hydrated high calcium lime the majority of the free lime, which is defined as the calcium oxide and calcium hydroxide that is not combined with other constituents, should be present as calcium hydroxide. British Standard 890 requires a minimum free lime and magnesia content (CaO + MgO), of 65 per cent.

Quicklime has a much higher bulk density than hydrated lime and it can be produced in various aggregate sizes. It is less dusty than hydrated lime but the dust is much more dangerous and ***strict safety precautions*** are necessary when it is used. For quicklime, British Standard 890 requires a minimum free lime and magnesia content, (CaO + MgO), of 85 per cent. ASTM C977 requires 90 per cent for both quicklime and hydrated lime.

Quicklime is an excellent stabiliser if the material is very wet. When it comes into contact with the wet soil the quicklime absorbs a large amount of water as it hydrates. This process is exothermic and the heat produced acts as a further drying agent for the soil. The removal of water and the increase in plastic limit cause a substantial and rapid increase in the strength and trafficability of the wet material.

In many parts of the world, lime has been produced on a small scale for many hundreds of years to make mortars and lime washes for buildings. Different types of kilns have been used and most appear to be relatively effective. Small batch kilns have subsequently been used to produce lime for stabilised layers on major road projects.

7.5.3 Selection of Lime Content

The procedure for selecting the lime content follows the steps used for selecting cement content and should, therefore, be carried out in accordance with British Standard 1924. The curing period for lime-stabilised materials is 21 days of moist cure followed by 7 days of soaking. If the amount of lime exceeds the ICL, the stabilised material will generally be non-plastic or only slightly plastic.

The temperature of the samples should be maintained near the ambient temperature. Accelerated curing at higher temperatures is not recommended because the correlation with normal curing at temperatures near to the ambient temperature can differ from soil to soil. At high temperatures the reaction products formed by lime and the reactive silica in the soil can be completely different from those formed at ambient temperatures.

7.6 Construction

7.6.1 General methodology

The construction of stabilised layers follows the same procedure whether the stabilising agent is cement, lime or mixtures of lime and pozzolan. After the surface of the layer has been shaped, the stabiliser is spread and then mixed through the layer. Sufficient water is added to meet the compaction requirements and the material mixed again. The layer must be compacted as soon as possible, trimmed, re-rolled and then cured. The effect of each operation on the design and performance of the pavement is discussed below.

Spreading the stabiliser

The stabiliser can be spread manually by ‘spotting’ the bags at predetermined intervals, breaking the bags and then raking the stabiliser across the surface as uniformly as possible. Lime has a much lower bulk density than cement and it is possible to achieve a more uniform distribution with lime when stabilisers are spread manually. Alternatively, mechanical spreaders can be used to meter the required amount of stabiliser onto the surface.

Mixing

Robust mixing equipment of suitable power for the pavement layer being processed is capable of pulverising the soil and blending it with the stabiliser and water. The most efficient of these machines carry out the operation in one pass, enabling the layer to be compacted quickly and minimising the loss of density and strength caused by any delay in compaction.

Multi-pass machines are satisfactory provided the length of pavement being processed is not excessive and each section of pavement can be processed within an acceptable time.

Graders have been used to mix stabilised materials but they are inefficient for pulverising cohesive materials and a considerable number of passes are needed before the quality of mixing is acceptable. They are therefore very slow and should only be considered for processing lime-stabilised layers because of the greater time available for doing so.

Plant pre-mixing gives the possibility of better control than in-place spreading and mixing provided that the plant is close enough to the site to overcome possible problems caused by delays in delivery. This can often be justified by the lower safety margins on stabiliser content and target layer thicknesses that are possible.

Compaction

A stabilised layer must be compacted as soon as possible after mixing has been completed in order that the full strength potential can be realised and the density can be achieved without over stressing the material. If the layer is over stressed, shear planes will be formed near the top of the layer and premature failure along this plane is likely, particularly when the layer is only covered by a surface dressing.

Multi-layer construction

When two or more lifts are required to construct a thick layer of stabilised material, care must be taken to prevent carbonation at the surface of the bottom lift. It is also important that the stabiliser is mixed to the full depth of each layer. A weak band of any type can cause over stressing and premature failure of the top lift followed by deterioration of the lower section.

In general, the thickness of a lift should not be greater than 200 mm or less than 100 mm. Care should be taken to reduce the density gradient in the layer because permeable material in the lower part of the layer makes it more susceptible to carbonation from below. If necessary, a layer should be compacted in two parts to make the bottom less permeable. The compaction operation should be completed within two hours and the length of road which is processed at any time should be adjusted to allow this to be achieved.

Curing

Proper curing is very important for three reasons:

i) It ensures that sufficient moisture is retained in the layer so that the stabiliser can continue to hydrate.
ii) It reduces shrinkage.
iii) It reduces the risk of carbonation from the top of the layer.

In a hot and dry climate the need for good curing is very important but the prevention of moisture loss is difficult. If the surface is sprayed constantly and kept damp day and night, the moisture content in the main portion of the layer will remain stable but the operation is likely to leach stabiliser from the top portion of the layer.

If the spraying operation is intermittent and the surface dries from time to time (a common occurrence when this method is used) the curing will be completely ineffective.

Spraying can be a much more efficient curing system if a layer of sand, 30 to 40 mm thick, is first spread on top of the stabilised layer. If this is done the number of spraying cycles per day can be reduced and there is a considerable saving in the amount of water used. After seven days, the sand should be brushed off and the surface primed with a low viscosity cutback bitumen. An alternative method of curing is to first apply a very light spray of water followed by either a viscous cutback bitumen, such as MC 3000, or a slow setting emulsion. Neither of these will completely penetrate the surface of the stabilised layer and will leave a continuous bitumen film to act as a curing membrane. It is essential that all traffic is kept off the membrane for seven days. After this time, any excess bitumen can be absorbed by sanding the surface. A prime coat cannot serve as a curing membrane because it penetrates too far into the layer and insufficient bitumen is retained on the surface to provide the necessary continuous film.

7.6.2 Control of shrinkage and reflection cracks

There is no simple method of preventing shrinkage cracks occurring in stabilised layers, however, design and construction techniques can be adopted which go some way to alleviating the problem. Shrinkage, particularly in cement-stabilised materials, has been shown to be influenced by

i) Loss of water, particularly during the initial curing period.
ii) Cement content.
iii) Density of the compacted material.
iv) Method of compaction.
v) Pre-treatment moisture content of the material to be stabilised.

Proper curing is essential not only for maintaining the hydration action but also to reduce volume changes within the layer. The longer the initial period of moist cure the smaller the shrinkage when the layer subsequently dries. When the layer eventually dries, the increased strength associated with a high stabiliser content will cause the shrinkage cracks to form at increased spacing and have substantial width.

With lower cement contents, the shrinkage cracks occur at reduced spacing and the material will crack more readily under traffic because of its reduced strength. The probability of these finer cracks reflecting through the surfacing is reduced, but the stabilised layer itself will be both weaker and less durable.

In order to maximise both the strength and durability of the pavement layer the material is generally compacted to the maximum density possible. However, for some stabilised materials it is sometimes difficult to achieve normal compaction standards and any increase in compactive effort to achieve them may have the adverse effect of causing shear planes in the surface of the layer or increasing the subsequent shrinkage of the material as its density is increased. If it proves difficult to achieve the target density, a higher stabiliser content should be considered in order that an adequately strong and durable layer can be produced at a lower density.

Laboratory tests have shown that samples compacted by impact loading shrink considerably more than those compacted by static loading or by kneading compaction. Where reflection cracking is likely to be a problem, it is therefore recommended that the layer should be compacted with pneumatic-tyred rollers rather than vibrating types.

Shrinkage problems in plastic gravels can be substantially reduced if air-dry gravel is used and the whole construction is completed within two hours, the water being added as late as possible during the mixing operation. It is generally not possible to use gravel in a completely air-dry condition, but the lower the initial moisture content and the quicker it is mixed and compacted, the smaller will be the subsequent shrinkage strains.

Having accepted that some shrinkage cracks are inevitable in the stabilised layer, the most effective method of preventing these from reflecting through the bituminous surfacing is to cover the cemented layer with a substantial thickness of granular material. This is the design philosophy in Charts B and D (Chapter 10). When cemented material is used as a roadbase (Chart F) a flexible surfacing such as a double surface dressing is recommended. Experience has shown that a further surface dressing applied after 2-3 years can partially or completely seal any subsequent cracking, particularly where lime is the stabilising agent.

7.6.3 Carbonation

If cement or lime-stabilised materials are exposed to air, the hydration products may react with carbon dioxide thereby reducing the strength of the material. This reaction is associated with a decrease in the pH of the material from more than 12 to about 8.5. The presence and depth of carbonation can be detected by testing the pH of the stabilised layer with phenolphthalein indicator and checking for the presence of carbonates with hydrochloric acid. A reasonable indication of whether the material being stabilised will be subject to serious carbonation can be obtained from the wet/dry test for durability. Good curing practices, as outlined above, are the best means of preventing carbonation in roadbases. The risk of carbonation can be reduced by taking the following precautions:

i) Avoid wet/dry cycles during the curing phase.
ii) Seal as soon as possible to exclude carbon dioxide.
iii) Compact as early as possible to increase the density and to reduce the permeability.
iv) Reduce the possibility of reflection cracks.

There may be some conflict between the last two points and care should be taken not to over compact the layer. Checks should be made during construction and if the depth of carbonated material is more than 2 to 3 mm the carbonated layer should be removed by heavy brushing or grading before the surfacing is applied.

7.6.4 Quality Control

A high level of quality control is necessary in the manufacture of cement and lime-stabilised materials, as with all other materials used in the road pavement, but several factors need special consideration.

Storage and handling of stabilisers

Unless cement and lime are properly stored and used in a fresh condition the quality of the pavement layer will be substantially reduced. Cement must be stored in a solid, watertight shed and the bags stacked as tightly as possible. Doors and windows should only be opened if absolutely necessary. The cement which is delivered from the manufacturer first should also be used first. Even if cement is properly stored the following losses in strength will occur:-

- After 3 months 20% reduction
- After 6 months 30% reduction

- After 1 year 40% reduction

Lime should be packed in sealed bags, tightly stacked and stored under cover or at least under a watertight tarpaulin. If it becomes contaminated or damp, it can only be used as a filler. Lime which is older than 6 months should be discarded.

Distribution of stabiliser

After the layer has been properly processed, at least 20 samples should be taken for determination of the stabiliser content. The mixing efficiency is acceptable if the coefficient of variation is less than 30 per cent. Great care is necessary in multi-layer construction to ensure that good mixing extends to the full depth of all the layers.

Opening to traffic

Insufficient research has been carried out to determine the precise effects of opening a road to traffic before the completion of the curing period but it is considered that allowing traffic on the pavement during the first two days can be beneficial for some stabilised layers provided the traffic does not mark the 'green' surface and ***all*** traffic is kept off the pavement from the end of the second day until one week has elapsed. Early trafficking has a similar effect to that of pre-cracking the layer by rolling within a day or two of its construction but rolling is preferred because it ensures even coverage of the full width of the carriageway. Layers which are pre-cracked or trafficked early must be allowed to develop sufficient strength to prevent abrasion of the edges of each crack before the layer is opened to general traffic. The slab strength of these layers is effectively destroyed and it is recommended that early trafficking is only acceptable for layers of cemented roadbase type CB2.

8 BITUMEN-BOUND MATERIALS

8.1 Introduction

This Chapter gives guidance on the design, manufacture and construction of hot premixed bituminous pavement materials (hot mix asphalt or HMA). Mix requirements are described for different traffic loading categories including severely loaded sites such as climbing lanes.

An important aspect is that it addresses the actual modes of failure that occur in HMA surfacings in tropical and sub-tropical environments and which often differ from those prevalent in cooler climates.

Figure 8.1 shows the layers which may be present in a road pavement and which may be bound with bitumen. Where thick HMA surfacing layers are required, they are normally constructed with a ‘wearing course’ laid on a ‘binder’ course.

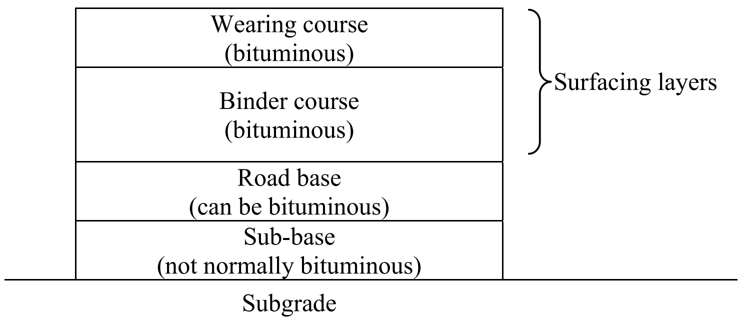


Figure 8-1 Pavement layers which may be bitumen bound

HMA wearing courses are the most critical layer in a pavement structure and must be of high quality and have predictable performance. 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.

Designing a mix having all of these characteristics will often result in conflicting design requirements. 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.

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.

The properties of mixes are very sensitive to their composition and to achieve the most satisfactory combination of the properties listed above for the many different design situations that are encountered requires a high degree of expertise and quality control.

8.2 Composition of HMA

8.2.1 Components of a mix

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.

8.2.2 Types of HMA in common use

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)

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 particle density after compaction but adjusted slightly to make room for sufficient bitumen. 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

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 manual.

Recipe specifications and the necessary compliance testing are simple to use and to implement, but the *direct* transfer of recipe designs between countries having different climates, materials and traffic loading characteristics is not recommended because there is no simple procedure for adequately assessing the effects of these differences. A local research programme is required to determine the best recipe methods. However, most authorities prefer to have a test procedure that they believe ensures satisfactory performance at all times and hence recipe specifications are not commonly used.

Hot Rolled Asphalt (HRA)

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.

Unlike AC mixes, the aggregate particle size distribution in HRAs is discontinuous and is referred to as being ‘gap-graded’. It is primarily 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. Rounded sand is not suitable and the lack of good angular sand is sometimes a problem.

Hot Rolled Asphalt (HRA) has been used extensively on heavily trafficked roads in Britain over many years and also in modified forms in other countries. In Australia HRA is recommended for residential streets because the mix has good workability and it is easy to achieve an impermeable layer.

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. However, 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.

HRA is also particularly suitable for use as a thin, flexible and durable wearing course for the lower traffic levels. The overall pavement design that utilises such a mix in a thin layer is such that the HMA surfacing behaves in what is termed ‘the controlled strain mode’ and, provided the road is designed and constructed properly, the thin surfacing is not required to withstand the high tensile strains that lead to fatigue. However, as the severity of traffic loading has increased, there has been a significant increase in the incidence of rutting in HRA. As a result this type of mix is not now recommended for some uses. For example, experience in the Republic of South Africa has shown that a thin continuously graded HMA surfacing (e.g. the nominal 9.5mm maximum stone size in Table 8.7) can work well and is the favoured option (see Chart G). No evidence of ‘bottom-up’ fatigue cracking has been reported but it should be noted that construction and mix design are carried out to high standards and the importance of providing a very stiff supporting structure beneath the thin continuously graded asphalt concrete cannot be over emphasised. Thus a strong aggregate roadbase is required and a stabilised sub-base is preferred.

Other types of mixes

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. For rehabilitation design the reader should refer to ERA’s Pavement Rehabilitation and Overlay Design Manual.

8.3 Factors affecting HMA design

8.3.1 Modes of failure of HMA surfacings

HMA must be designed to resist three main modes of deterioration. These are;

(i) cracking;

(ii) plastic deformation; and

(iii) loss of surfacing aggregate.

Cracking in HMA surfacings

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, Dauzats and Linder, 1982). It is now generally accepted that ‘top down’ cracking occurs in many countries including those with more temperate climates. It is, for example, the main mode of failure on the UK’s heavily trafficked network (Nunn et al, 1997).

‘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 8.2 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 4.8 to 6.7 log poises (or $7.5x10^4$ to $5x10^6$ poises).

One way of improving the durability of HMA is to increase the bitumen content. This reduces the air void content and the rate of oxidation and any surface cracking in the 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.

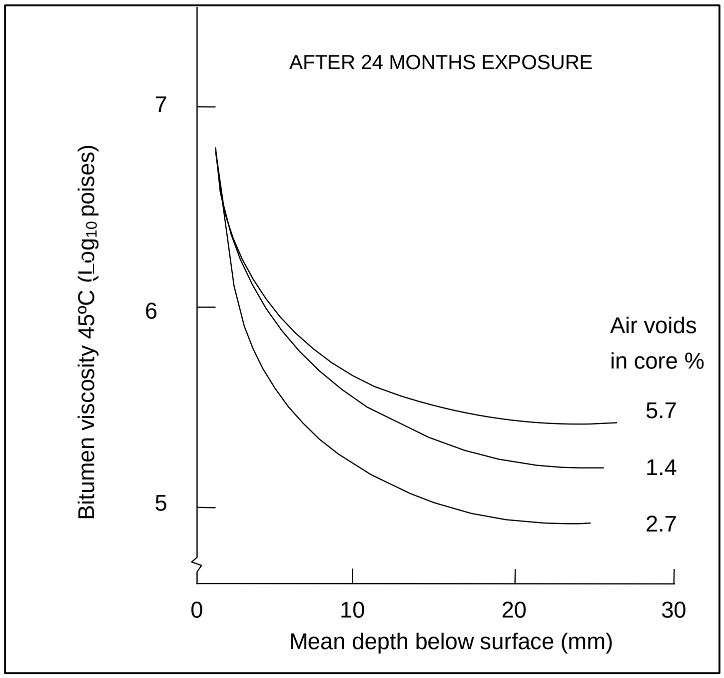


Figure 8-2 Bitumen viscosity versus depth in cores taken from a road site

Failure of asphalt surfacings by plastic deformation

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 under estimate of the degree of secondary compaction that occurs under heavy traffic which reduces the air voids content (i.e. voids in the mix, VIM) to a critical level at which plastic deformation occurs relatively rapidly.

The relationship between in situ VIM and asphalt deformation observed on severely loaded sites in countries experiencing high road temperatures is illustrated in Figure 8.3. 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, 1994).

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. Typical relationships between the rate of reduction in VIM and traffic on a climbing lane are shown in Figure 8.4. Mix 1 will perform well but the other mixes will deform.

The Asphalt Institute recommends that the design bitumen content is that which gives 4 per cent VIM at the appropriate laboratory design level of compaction. However, the level of compaction specified for construction is, typically, 96 per cent of the laboratory design density therefore the initial in situ VIM will be approximately 8 per cent. However, where AC surfacings with VIM of 8 per cent are laid in tropical environments they will be very vulnerable to premature deterioration through ageing of the bitumen and ‘top down’ cracking. In addition, if a mix is vulnerable to densification to less than 3 per cent VIM under heavy traffic, then simply compacting it to 8 per cent VIM during construction may not prevent this from happening.

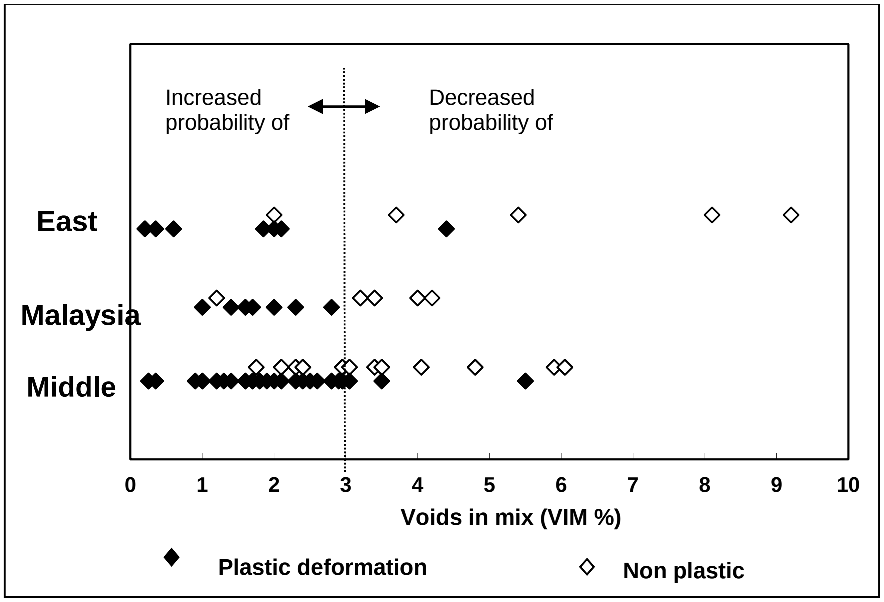


Figure 8-3 Occurrence of plastic deformation in AC wearing courses

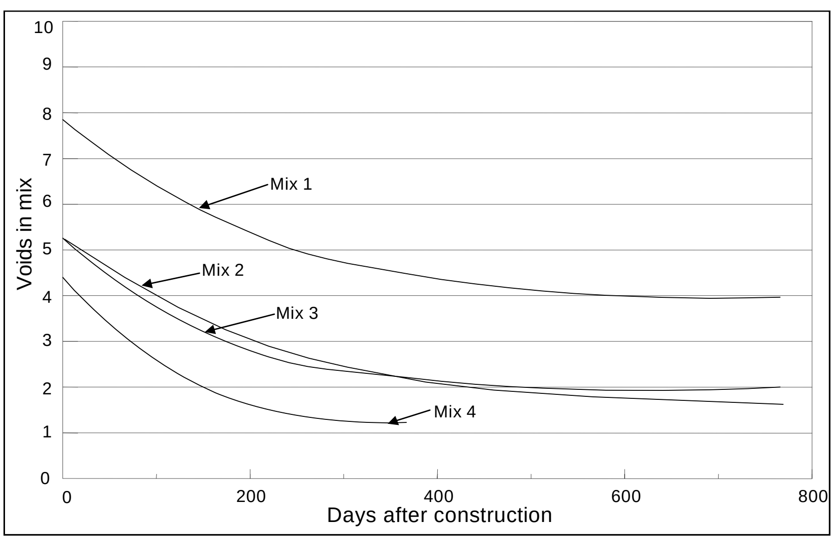


Figure 8-4 Reduction in VIM in the wheel path of AC wearing courses

(designed by Marshall procedure)

In summary, if the initial VIM is high and traffic loading is not heavy, then the rate of reduction in VIM may be slow enough to allow appreciable bitumen hardening to take place. This will result in an increase in resistance to densification and deformation. In contrast, heavy and intense early trafficking is likely to reduce VIM rapidly and prevent appreciable bitumen hardening in the body of the AC. Mix stiffness may not then increase sufficiently to prevent a critical reduction in VIM and subsequent deformation. The design of HMA therefore requires a critical balance between the required properties for the traffic and operating conditions for each project.

Loss of surfacing aggregate, or fretting

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

(i) 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) 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.

8.3.2 Effects of vehicle characteristics

Axle loads and vehicle speeds

Traffic loading for pavement design purposes is expressed in terms of equivalent standard axles (ESAs). 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 is important, it is the characteristics and pressure of the tyres that have most influence on the performance of HMA surfacing layers.

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

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 wheel paths. 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 wheel path. However, radial ply tyres tend to run in the bottom of the ruts thereby producing much narrower wheel paths 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

Tyre pressures have also increased significantly over recent years and this has resulted in more severe loading at the road surface. The tyre pressure of 0.48 MPa (70 psi) that were used at the AASHO Road Test in 1962 are low compared with the pressures used by the operators of heavy vehicles nowadays; over 90% of heavy trucks will have higher tyre pressures. Mean values of more than 0.7 MPa (102 psi) are common and some of the very heavily loaded trucks can have tyre pressures of up to 1.03MPa (150 psi).

In summary, the operating conditions of road surfaces are more severe than in the past and greater control of the quality of the HMA is required to achieve good performance.

8.3.3 Road Maintenance

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.

It is 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.

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 a ‘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 treatment must take into account the hardness of the new HMA surfacing (Chapter 9). A short delay may be necessary to allow the surfacing to harden before the dressing is applied.

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. For full details of maintenance and rehabilitation options refer to ERA’s *Pavement Rehabilitation and Overlay Design Manual.*

8.3.4 Safety Considerations

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.

8.4 Materials for HMA

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

8.4.1 Aggregates

Aggregate is the major component in HMA and the quality and physical properties of this material has a large influence on mix performance. 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 from particles having a coarse micro texture.

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.

The aggregate should have the following characteristics;

(i) Be angular and not excessively flaky, to provide good mechanical interlock;
(ii) Be clean and free of clay and organic material;
(iii) Be strong enough to resist crushing during mixing and laying as well as in service;
(iv) Be resistant to abrasion and polishing when exposed to traffic;
(v) Be non absorptive - highly absorptive aggregates are wasteful of bitumen and also give rise to problems in mix design; and
(vi) 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.

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.

The required properties for aggregates are given in Table 8.1 and summaries of the relevant test methods are given in Appendix A.

In the UK detailed specifications have been developed for the Polished Stone Value required at sites which present different degrees of risk. These specifications are reproduced in Table 8.2.

Table 8-1 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 >1.5 x 10^6 ESA	 > 35 > 40	
	(Material passing 0.425mm sieve) Plasticity Index[2] Linear shrinkage %	 < 4 < 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 cv/lane/day > 3250 cv/lane/day	 < 16 < 12	 - -
Polishing	Polished Stone Value [4]	(see Table 8.2)	-
Water absorption	Water absorption [6]	<2	
Soundness [7] (5 cycles, % loss)	Sodium Sulphate Test: Coarse Fine Magnesium Sulphate Test: Coarse Fine	 < 10 < 16 < 15 < 20	
Bitumen affinity	Immersion Mechanical Test: Index of retained Marshall stability Static Immersion Test [8] Retained Indirect Tensile strength [9]	> 75 > 95% coating retained > 79% (at 7%VIM)	

Notes:
1. AASHTO T176-86
2. British Standard 1377: Part 2 (1990)
3. British Standard 812, Part 105 (1990)
4. British Standard 812, Part 3 (1985)
5. ASTM C131 and C535
6. British Standard 812, Part 2 1975
7. AASHTO T104-99
8. D Whiteoak (1990)
9. AASHTO T283

Table 8-2 Minimum PSV for Coarse Surfacing Aggregates

<table>
<tr><th rowspan="2">Site definition</th><th colspan="14">Traffic (cv[1]/l/d) at design life</th></tr>
<tr><th>0 - 100</th><th>101 - 250</th><th>251 - 500</th><th>501 - 750</th><th>751 - 1000</th><th>1001-1250</th><th>1251-1500</th><th>1501-1750</th><th>1751-2000</th><th>2001-2250</th><th>2251-2500</th><th>2501-2750</th><th>2751-3250</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>

Note. cv = commercial vehicles defined as those over 15 kN unladen weight

8.4.2 Bitumen for HMA

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

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) (Whiteoak, 1990) which is illustrated in Appendix B.

Several authorities now produce alternative specifications based on viscosity. Suitable apparatus for measuring viscosity may not be readily available 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

Bitumen samples should be tested in both the ‘as delivered state’ and also after pre-hardening, which is intended to simulate the ageing of 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. 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

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 8.3.

Table 8-3 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

Requirements for viscosity graded bitumens

Authorities such as AASHTO, ASTM, the Standards Association of Australia 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 8.4.

Table 8-4 South African Specifications for Penetration Grade Bitumens (SANS-307)

Property	Requirement				Test Method
	40/50	**60/70**	**80/100**	**150/200**	
Penetration @ 25ºC/100 g/5s, 1/10 mm	40-50	60-70	80-100	150-200	ASTM D5-IP49
Softening Point (ring and ball) ºC	49-59	46-56	42-51	36-43	ASTM D36
Viscosity @ 60ºC, Pa.s	220-400	120-250	75-150	30-60	ASTM D 4402
Viscosity @ 135ºC, Pa.s	0.27-0.65	0.22-0.45	0.15-0.40	0.12-0.30	ASTM D 4402
Spot test, % xylene, max	30	30	30	30	AASHTO T102
	Performance after rolling thin filmed oven test:				ASTM D 2872
a) Mass change, % (m/m) max	0.5	0.5	0.5	0.5	ASTM D 2872
b) Viscosity @ 60ºC, % of original max	300	300	300	300	ASTM D 4402
c) Softening point, ºC min	52	48	44	37	ASTM D 36
d) Increase in softening point, ºC max	9	9	9	11	ASTM D 36
e) Retained penetration, % of original min	60	55	50	50	ASTM D5-IP49

European specifications for paving grade bitumens

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 standard includes 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. Specifications for grades most appropriate for use in tropical countries have been selected from the standards and reproduced in Table 8.5.

Table 8-5 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						
Flash point	EN 22592	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										

Bitumen durability

Bitumens derived from different sources of crude oil can have varying resistance to ageing and 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, fuel oils 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.

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. Financial restraints may mean that authorities must purchase bitumen at the most competitive open market rates. However, the import of more durable bitumen should be seriously considered for major projects such as international airfields and important transcontinental routes.

8.5 Mix Design for HMA

8.5.1 Introduction to mix design methods

Ideally the design of an HMA mix involves the following iterative process;

(i) Establish 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) Adjust the mix composition and re-test until the design requirements are satisfied.

Mix design for AC surfacing materials is commonly based on the recommendations given in the Asphalt Institute Manual Series, MS-2, 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 described in Appendix C.

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 in the design process 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.

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 design (Asphalt Institute Superpave Series No.1 and No.2, 1996, 2001 and Appendix D). This procedure uses a gyratory compactor for sample preparation which produces incremental steps of compactive effort. The levels of compaction used in the design are related to the expected pavement temperature and traffic loading. For traffic loading up to 1 million esa, mix composition is essentially based on the principles given in MS-2. For design traffic between 1 and 10 million esa an intermediate level of performance testing is called for and detailed analysis is required for traffic loading in excess of 10 million esa.

The cost of the full suite of Superpave test equipment is substantial and a mix design requiring the input of qualified technicians can take as long as one month to complete. An outline of the Superpave method is given in Appendix D.

AUSTROADS have developed a simpler 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.

In South Africa a combination of Marshall, Creep and Indirect Tensile Strength test requirements are specified for design traffic categories up to 50 million ESA (TRH 8:1987). However, the South African interim guideline for design of Hot Mix Asphalt (2001) and supported by the user guide for the design of Hot Mix Asphalt (SABITA manual 24, 2005) based on the new performance based mix design procedures being considered for adoption are available from SABITA.

The difference in costs between carrying out designs using the Marshall method with an automatic compaction hammer and the complete Superpave test method is very large. Marshall design can be carried out in site laboratories on most major projects but it is impossible for this to be done with Superpave designs. It is expected that the Marshall procedure will remain the principle method of mix design for AC mixes in many countries for many years and the use of performance tests of the types described in Appendix F is also encouraged.

8.5.2 Volumetric design of HMA mixes

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. This 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.

A representation of volumes in a compacted bituminous mixture is shown in Figure 8.2. The basic definitions used in volumetric design are summarised here. More detail is contained in Appendix C.

Air Voids (VIM)

This 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 (see Figure 8.5).

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)

This is the volume of void space between the aggregate particles of a compacted paving mixture. It is the sum of VIM and Pbe expressed as a percent of the total volume of the sample.

Voids Filled with Bitumen (VFB)

This is the portion of the volume of void space between the aggregate particles (VMA) that is occupied by the effective bitumen.

To determine volumes, the specific gravities of the mix components must also be determined. These are defined as shown in Appendix C. The nomenclature and test methods for volumetric design are summarised in Table 8.6.

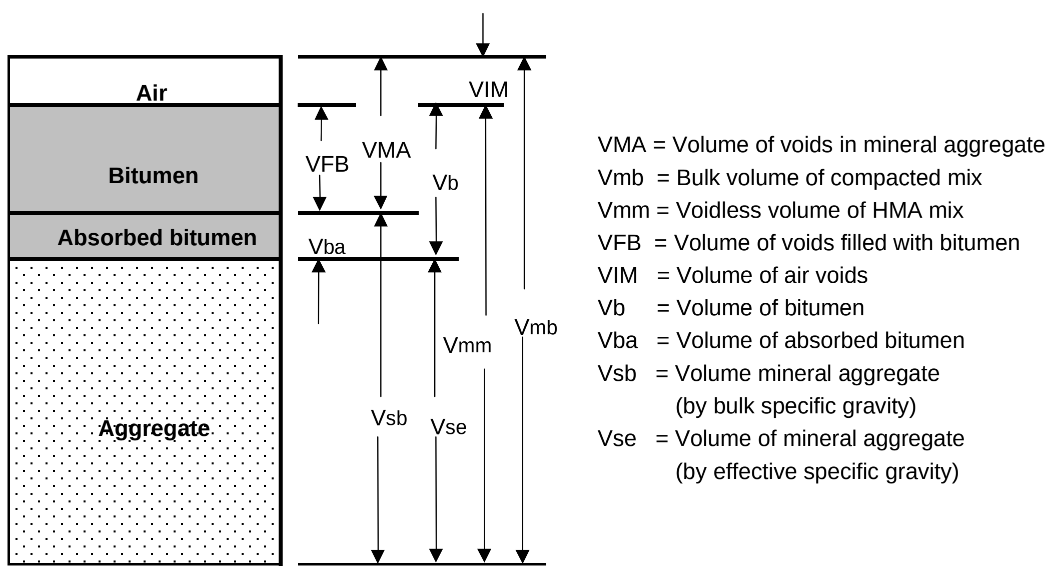


Figure 8-5 Representation of volumes in a compacted HMA specimen

(Asphalt Institute, MS-2, 1994)

Table 8-6 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	-	-

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

8.5.3 Aggregate particle size distribution for HMA

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.

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.

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.

The nominal maximum stone size determines the minimum VMA required in the aggregate blend and the maximum stone size that can be used in a mix is governed by the proposed thickness of the HMA layer. To achieve good compaction the layer thickness will normally have to be between 2 x the maximum stone size for fine mixes and 4 x the maximum stone size for mixes with a high content of coarse aggregates such as those normally recommended for severe traffic loading.

Particle size distributions for AC wearing courses

Authorities will often base the choice of particle size distribution on local experience or the recommendations of the Asphalt Institute. Particle size distributions recommended by the Asphalt Institute for wearing course layers are shown in Table 8.7.

Table 8-7 Particle Size Distributions for AC Wearing Courses

(Asphalt Institute, 1994)

Sieve size (mm)	Nominal maximum stone size (mm) Percentage passing sieve		
	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
0.150	-	-	-
0.075	2 – 8	2 –10	2 – 10
Bitumen content (%)	4 - 10	4 - 11	5 - 12

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 differs by more than 0.2 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.

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.

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 through which the distribution must not pass. 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 8.6. 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.

Use of the restricted zone must be treated with care. 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 use in special circumstances.

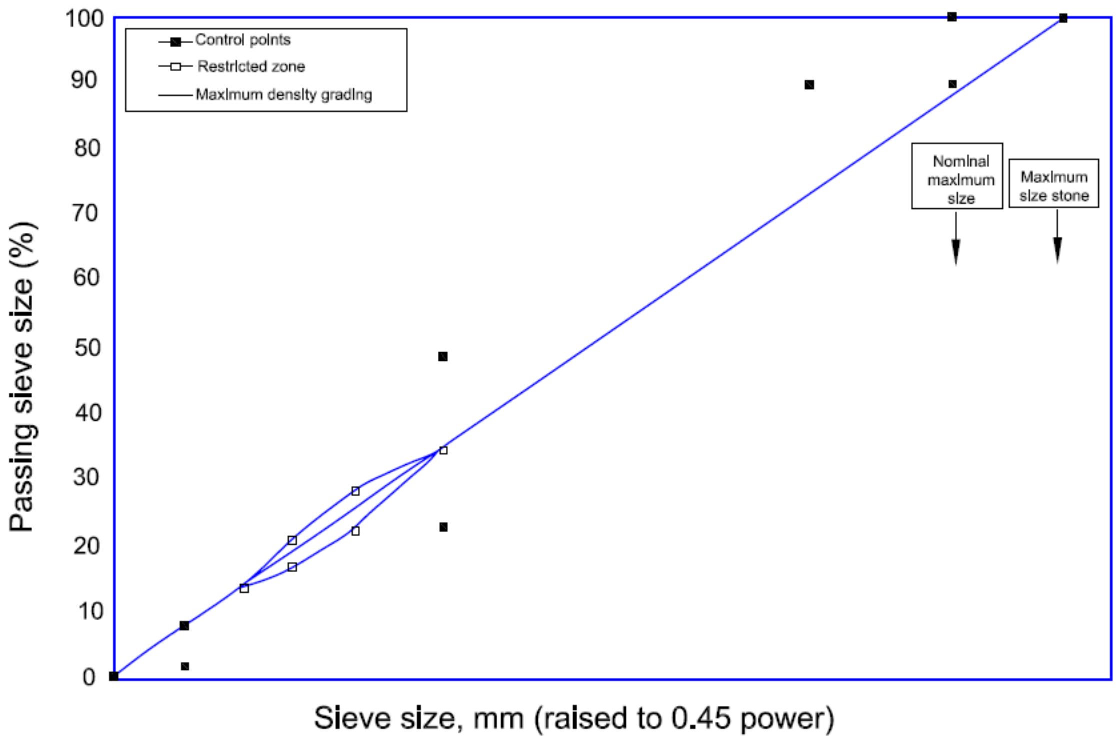


Figure 8-6 Example of generalised Superpave™ particle size distribution

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

The specified control points and restricted zones for HMA wearing course mixes depend on the nominal maximum aggregate size. Superpave particle size distributions and examples of complete charts are given in Appendix D.

Particle size distributions for AC binder courses and road bases

Neither the Asphalt Institute's MS-2 (1994) nor Superpave (1996) describes particle size distributions specifically for binder courses and road bases. 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 above).

Some of the particle size distributions recommended by the Asphalt Institute are identified as being suitable for binder course and road base. These are shown in Tables 8.8 and 8.9. Because the Marshall test method cannot be used to design mixes with aggregate larger than 25mm, the design of coarse binder courses and road bases tends to rely on empirical knowledge.

Table 8-8 Particle Size Distributions for AC Roadbases and Binder Courses

(Asphalt Institute, 1994)

Layer	**Roadbase**	**Binder course**
Sieve size (mm)	**Nominal maximum size (mm)**	
	Per cent passing sieve size	
	37.5	25
50	100	
37.5	90 – 100	100
25	–	90 – 100
19	56 – 80	–
12.5	–	56 – 80
9.5	–	–
4.75	23 – 53	29 – 59
2.36	15 – 41	19 – 45
1.18	–	–
0.600	–	–
0.300	4 – 16	5 – 17
0.150	–	–
0.075	0 – 6	1 - 7
Typical bitumen content (%)	3 - 8	3 - 9

Particle size distribution for Dense Bitumen Macadam (DBM)

Particle size distributions recommended for DBM wearing courses and for binder course and roadbase layers are shown in Tables 8.9 and 8.10.

Table 8-9 Particle Size Distributions for DBM Wearing Courses

Sieve size (mm)	**UK nomenclature for size (mm)**	
	Percentage passing sieve size	
	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

These mixes have traditionally been made to recipes but the wearing course mixes can 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 8-10 Particle Size Distributions for DBM Binder Course and Roadbase Layers

Sieve size (mm)	Binder course	Roadbase	
	UK nomenclature for size (mm)		
	Percentage passing sieve size		
	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%

8.6 Mix Design Specifications

8.6.1 Mix design for continuously graded wearing courses

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. This sensitivity is illustrated by the small range of VIM values shown in Table 8.11 and the effect they have on mix performance.

Table 8-11 Critical Values of VIM (after Secondary Compaction)

VIM in a wearing course material (per cent)	Effect
> 8	Extremely permeable to air and water. Oxidation of the bitumen very rapid in hot climates
> 5	Increasingly permeable to air and prone to oxidation of the bitumen
4 or 5	Target for design
3-5	For a durable and stable mix
< 3	Prone to plastic deformation under heavy loading

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 aggregate size as shown in Table 8.12.

Table 8-12 Minimum VMA Specified for AC Mixes

Nominal maximum aggregate size (mm)	Minimum VMA (%)	
	VIM = 4.0 %	VIM = 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

8.6.2 VMA and bitumen film thickness

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 aggregate 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.

Unfortunately the measurement of VMA is subject to large variability 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 8.12). Furthermore, two particle size distributions having different maximum sized aggregate but different aggregate types may have similar surface areas and hence require the same minimum VMA.

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 8 microns it is recommended that the determination of VMA is 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.

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

(i) < 5 million esa;

(ii) > 5 million esa; and

(iii) Severe sites are sites where heavy traffic is slow moving, accelerating or braking. These include steep gradients, climbing lanes and junctions. (For guidance, a steep gradient is defined as greater than 7 percent but anywhere where heavy trucks are reduced to speeds of less than 15 km/hour are severe sites.

The effect of VMA and of errors in the selection of the level of compaction that should be used is discussed further in Appendices C, D and E.

8.6.3 For design traffic less than 5 million ESA.

In principle any of the wearing course or binder course gradings described in Section 8.5 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 will have a coarser texture. All mixes should be designed to the Asphalt Institute (MS-2, 1994) Marshall criteria for wearing courses shown in Table 8.13.

Table 8-13 AC Wearing Course Specifications for up to 5 million esa

Category and design traffic (million ESA)	No. of blows of Marshall compaction hammer	Min. 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

8.6.4 For design traffic greater than 5 million esa

Local experience may justify higher design traffic loads than are specified here. However, such 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 8.6.6) and to design a suitable mix for these locations.

The Marshall requirements for mixes designed for this category of design traffic are summarised in Table 8.14.

Table 8-14 AC wearing course specification for more than 5 million esa

Category and design traffic (million ESA)	No. of blows of Marshall compaction hammer	Min. Stability (N)	Flow (mm)	VFB (%)	VIM at optimum bitumen content (%)
Very heavy (> 5)	75	9000	2 - 3.5	65-73	5

8.6.5 Other considerations for design of continuously graded mixes

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 8.7.

In the refusal density test (see section 8.6.6 and Appendix E) VMA tends to remain constant until the structure starts to become overfilled.

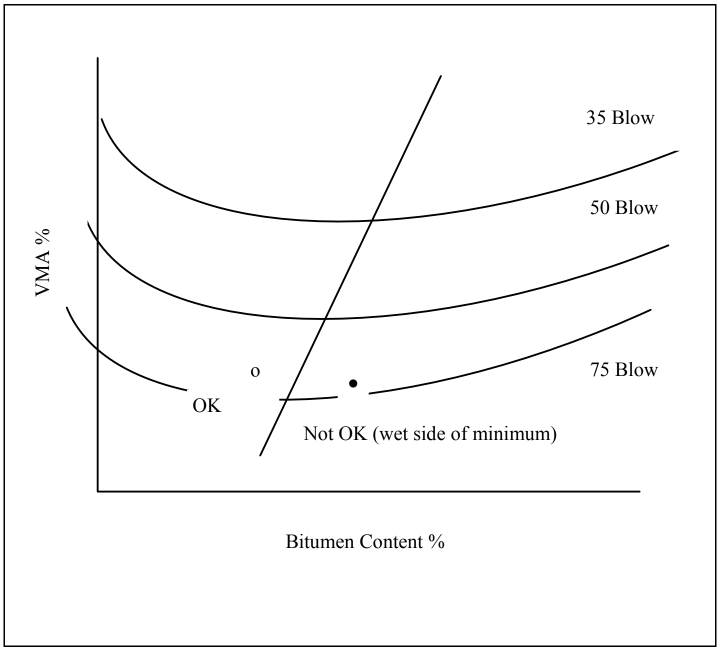


Figure 8-7 Effect of Marshall compactive effort on VMA and VIM (Asphalt Institute, 1994)

8.6.6 Mix design for severe sites

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 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.

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 8.8 and 8.10 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 8.14 for very heavy traffic.

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. The additional coarse aggregate should be from the same source as the aggregate used in the Marshall design.

The particle size distribution given in Table 8.8 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 8.10 will often be a good compromise.

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 8.8 and 8.10.

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) (Appendix E) which is also incorporated into a CEN Standard, prEN 12697-9).

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.

The design bitumen content is determined by compacting samples to refusal using the method described in Appendix E. The thickness of the compacted samples should be approximately the same as the compacted layer to be laid on the road. Samples should be made at the bitumen content which gives 6 per cent VIM in the Marshall test and at decreasing increments of 0.5 per cent until the bitumen content which gives the 3 per cent VIM at refusal density can be identified.

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 the VIM is 3 per cent. Pre-construction compaction trials are essential to the selection of the final mix design (see Appendix E).

8.6.7 Selection of grade of bitumen

60/70 penetration grade bitumen is generally recommended for HMA in hot climates. For severe sites the additional mix stiffness that should result from using 40/50 penetration grade bitumen may be justified. Typically an increase in mixing temperature of 10^0C will be necessary if the harder bitumen is used.

8.6.8 Use of recycled asphalt

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 G.

8.7 Mix Production

8.7.1 General requirements

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.

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.

(iv) Correct adjustment of dust extraction equipment.

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.

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.

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'.

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.

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 is discussed in Appendix C and typical plant mix tolerances are summarised in Table 8.15.

Table 8-15 Tolerances for the Manufacture of AC

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

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 particle size distributions will run approximately parallel with the boundaries of this envelope. When the HMA is being made to a Superpave particle size distribution avoiding the restricted zone, the working envelope may be further restricted.

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.

8.7.2 Aggregate stockpiles and cold feeds

The importance of good stockpile management and control of cold bin settings cannot be over stated. 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.

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.

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).

8.8 Construction of Asphalt Surfacings

The purpose of this Section 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 if comprehensive detail of construction methods is required.

8.8.1 Mixing and compaction

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.

Typical mixing temperatures are summarised in Table 8.16 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 be determined.

Table 8-16 HMA Mixing Temperatures

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

Thorough compaction during construction is vital because traffic is likely to give very little additional compaction outside of the wheel paths. 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.

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.

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.

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 may be effective when used at the lower end of the acceptable range of compaction temperatures.

8.8.2 VIM after construction

AC mixes designed by the Marshall method

It is necessary to lay AC wearing courses with VIM greater than 4 per cent to allow for compaction under traffic. As the bitumen in the surfacing hardens during the secondary compaction phase it will become stiffer, less temperature sensitive and more resistant to densification.

A typical requirement at the time of construction is that AC mixes should be compacted to at least 96 per cent of the design density. This means that a mix which has been designed to 4 per cent VIM can have up to 8 per cent VIM immediately after construction. In the past some authorities have encouraged a target VIM of 3 to 5 per cent immediately after compaction and this has resulted in contractors increasing the bitumen content in order to achieve the required densities, a practice which dramatically increases the risk of plastic deformation, especially for heavily trafficked roads.

There will be a considerable improvement in durability if an HMA layer can be compacted to higher densities at the design bitumen content, but whether this is possible is dependent on both the availability of effective modern compaction rollers and on the characteristics of the mix. It cannot be assumed that such higher densities can be easily and consistently achieved and compaction trials are essential to test this.

In Section 8.3.1 it was shown that in tropical climates rapid bitumen hardening can be expected in a mix which is laid with 8 per cent VIM, particularly outside the wheel paths where compaction by traffic will be limited. In Section 8.3.3 it was pointed out that early maintenance will be needed if top-down cracking is to be prevented. 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

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.

Mixes of this type should be very resistant to long term secondary compaction under traffic and virtually no densification will occur outside of the wheel paths. It is essential to seal these surfacings as part of the construction process to prevent ingress of water and premature ‘top down’ cracking. Sealing should be carried out as soon as surface hardness tests (Chapter 9 and Appendix J) show that there is sufficient resistance to chipping embedment. Cape seals and coarse textured slurry seals can be used as alternatives to surface dressings.

8.8.3 Segregation

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.

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

(i) Unnecessary movement of materials in the stockpiles;
(ii) Unnecessarily high drop heights from pug mills or hot storage bins;
(iii) Letting storage bins, whether of cold or hot materials, run too low;
(iv) Poor paving practice.

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 hoppers or storage bins are run low the risk of segregation is increased.

Reducing drop heights as far as is practical may help to prevent large aggregate particles from falling to the edge of the delivery lorry each time a batch is dropped from the pug mill.

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.

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.

Consideration should also be given to leaving the paver wings open during the days 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 segregated material.

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 aggregate size.

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 (see References).

9 SURFACE TREATMENTS

9.1 Introduction

This chapter is a guide to the design of surface treatments including Otta seals. It provides a method for the engineer to base specific decisions to suit particular local conditions and draws attention to some of the common mistakes that are made, thereby producing cost effective results. It also contains descriptions of other types of surface treatments.

A surface treatment is a simple, highly effective and inexpensive road surfacing if adequate care is taken in the planning and execution of the work. The process is used for surfacing both medium and lightly trafficked roads and also as a maintenance treatment for roads of all kinds.

A surface treatment comprises a thin film of bitumen which is sprayed onto the road surface and then covered with a layer of stone chippings. The thin film of binder acts as a waterproofing seal preventing the entry of surface water into the road structure. The stone chippings protect this film of binder from damage by vehicle tyres, and form a durable, skid-resistant and dust-free wearing surface. In many circumstances the process is repeated to provide a double or, less often, a triple layer of chippings.

A surface treatment is an effective and economical running surface for newly constructed road pavements capable of carrying traffic flows of up to 500 vehicles/lane/day and up to 1000 vehicles/lane/day and more if the roadbase is stable or if a suitable seal is used.

A correctly designed and constructed surface treatment should last at least five years before resealing with another surface treatment becomes necessary. If traffic growth over a period of several years necessitates a more substantial surfacing or increased pavement thickness, a bituminous overlay can be laid over the original surface treatment when the need arises.

A surface treatment is also a very effective maintenance technique which is capable of greatly extending the life of a structurally sound road pavement if the process is undertaken at the optimum time. Under certain circumstances a surface treatment may also retard the rate of failure of a structurally inadequate road pavement by preventing the ingress of water and preserving the inherent strength of the pavement layers and the subgrade.

An Otta seal differs from a surface treatment in that a graded gravel or crushed aggregate containing all sizes including filler is used instead of single-sized chippings. However, the method of construction is very different. An Otta seal can be either single or double.

For further details of the design and construction of surface treatments refer to ERA's Best Practice Manual for Thin Bituminous Surfacings.

9.2 Types of Surface Treatment

Surface treatments can be constructed in a number of ways to suit site conditions. The common types of surface treatments are illustrated in Figure 9.1.

9.2.1 Single Surface Treatment

A single surface treatment is ideal for use as a principal maintenance treatment on existing paved roads. It waterproofs the road surface, restores skid resistance and arrests deterioration.

On a new unbound roadbase a double surface treatment is considerably more durable than a single treatment and is normally recommended, however, a 'racked-in' dressing (see below) may be suitable for use on a new roadbase which has a tightly knit surface because of the heavier applications of binder which is used with this type of single dressing.

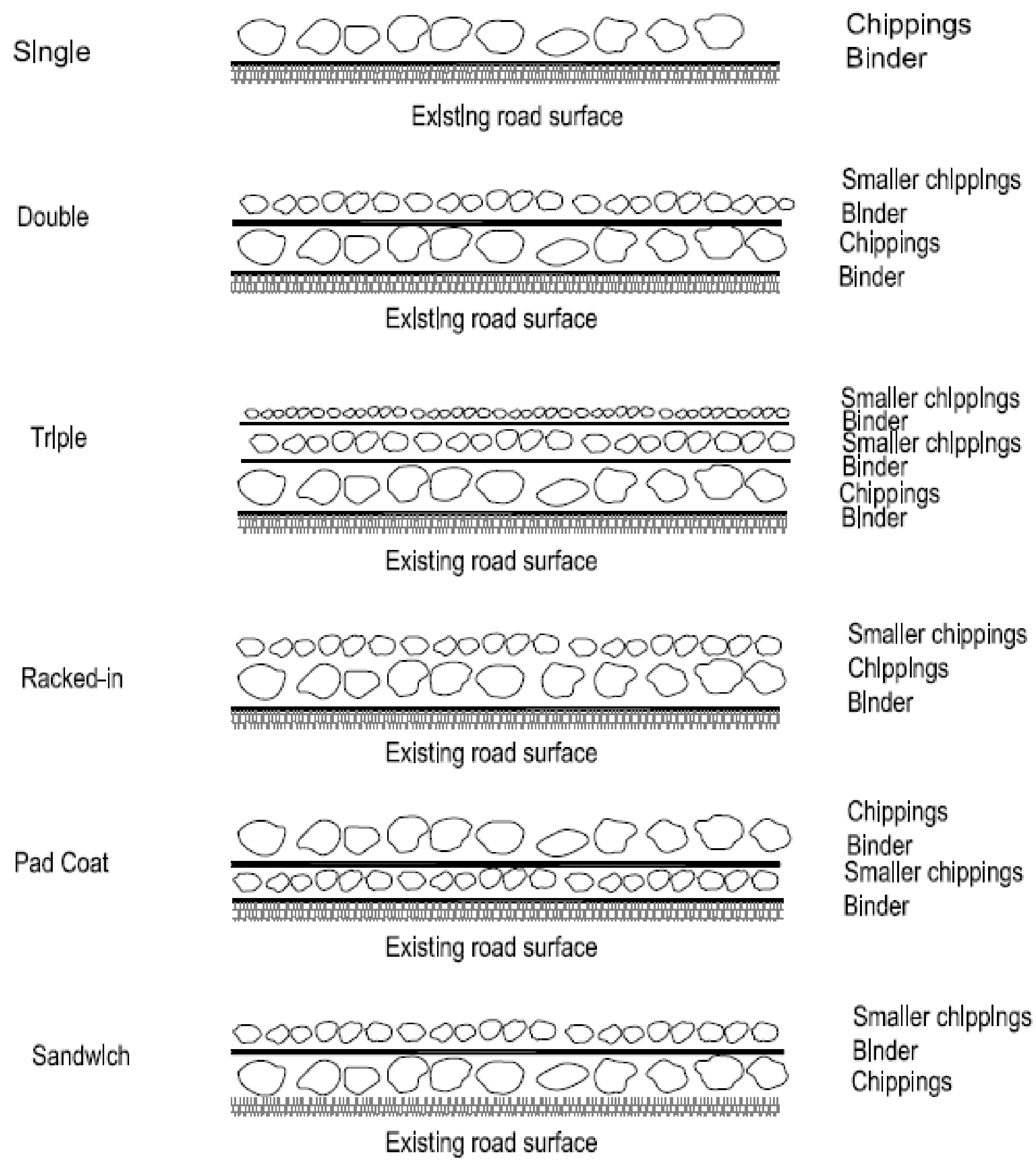


Figure 9-1 Types of Surface Treatments

9.2.2 Double Surface Treatment

Double surface treatments should be used when:

i) A new roadbase is surface treated.
ii) Extra 'cover' is required on an existing bituminous road surface because of its condition (e.g. when the surface is slightly cracked or patched).
iii) There is a requirement to maximize durability and minimize the frequency of maintenance and resealing operations.

The quality of a double surface treatment will be greatly enhanced if traffic is allowed to run on the first treatment for a minimum period of 2-3 weeks (and preferable longer) before the second treatment is applied. This allows the chippings of the first treatment to adopt a stable interlocking mosaic which provides a firm foundation for the second treatment. However, traffic and animals may cause contamination of the surface with mud or soil during this period and this must be thoroughly swept off before the second treatment is applied. Such cleaning is sometimes difficult to achieve and the early application of the second seal to prevent such contamination may give a better result.

Sand may sometimes be used as an alternative to chippings for the second treatment. Although it cannot contribute to the overall all thickness of the surfacing, the combination of binder and sand provides a useful grouting medium for the chipping of the first seal and helps to hold them in place more firmly when they are poorly shaped. A slurry seal may also be used for the same purpose (see below).

9.2.3 Triple Surface Treatment

A triple surface treatment may be used to advantage where a new road is expected to carry high traffic volumes from the outset. The application of a small chipping in the third seal will reduce noise generated by traffic and the additional binder will ensure a longer maintenance-free service life.

9.2.4 Racked-In Surface Treatment

This treatment is recommended for use where traffic is particularly heavy or fast. A heavy single application of binder is made and a layer of large chippings is spread to give approximately 90 per cent coverage. This is followed immediately by the application of smaller chippings which should 'lock-in' the larger aggregate and form a stable mosaic. The amount of bitumen used is more than would be used with a single seal but less than for a double seal. The main advantages of the racked-in surface treatment are:

Less risk of dislodged large chippings.

- Early stability through good mechanical interlock.
- Good surface texture.

9.2.5 Other Types of Surface Treatment

'Pad coats' are used where the hardness of the existing road surface allows very little embedment of the first layer of chippings, such as on a newly constructed cement stabilised roadbase or a dense crushed rock base. A first layer of nominal 6mm chippings will adhere well to the hard surface and will provide a 'key' for larger 10mm or 14mm chippings in the second layer of the treatment. 'Sandwich' surface treatments are principally used on existing binder-rich surfaces and sometimes on gradients to reduce the tendency for the binder to flow down the slope.

9.3 Chippings for Surface Treatments

The selection of chipping sizes is based on the volume of commercial vehicles having unladen weight of more than 1.5 tonnes and the hardness of the existing pavement. Ideally, chippings used for surface treatment should be single sized, cubical in shape, clean and free from dust, strong, durable, and not susceptible to polishing under the action of traffic. In practice the chippings available usually fall short of this ideal.

It is recommended that chippings used for surface treatment should comply with the requirements of Table 9.1 for higher levels of traffic, and to the requirements of Table 9.2 for lightly trafficked roads of up to 250 vehicles per day:

Table 9-1 Grading Limits, Specified Size and Maximum Flakiness Index for Surface Treatment Aggregates

Test Sieve (mm)	Nominal Size of Aggregates (mm)			
	20	**14**	**10**	**6.3**
28	100	-	-	-
20	85-100	100	-	-
14	0-35	85-100	100	-
10	0-7	0-35	85-100	100
6.3	-	0-7	0-35	85-100
5.0	-	-	0-10	-
3.35	-	-	-	0-35
2.36	0-2	0-2	0-2	0-10
0.600	-	-	-	0-2
0.075	0-1	0-1	0-1	0-1
	Minimum Percentage by Mass Retained on Test Sieve			
Specified Size	65	65	65	65
Max. Flakiness Index	25	25	25	-

Samples of the chippings should be tested for grading, flakiness index, aggregate crushing value and, when appropriate, the polished stone value and aggregate abrasion value. Sampling and testing should be in accordance with the methods described in Appendix A.

Specifications for maximum aggregate crushing value (ACV) for surface treatment chippings typically lie in the range 20 to 30. For lightly trafficked roads the higher value is likely to be adequate but on more heavily trafficked roads a maximum ACV of 20 is recommended.

The polished stone value (PSV) of the chippings is important if the primary purpose of the surface treatment is to restore or enhance the skid resistance of the road surface. The PSV required in a particular situation is related to the nature of the road site and the speed and intensity of the traffic. The resistance to skidding is also dependent upon the macro texture of the surface which, in turn, is affected by the durability of the exposed aggregate. Table

8.2 gives recommended values of PSV for various road and traffic conditions and provides and indication of the required aggregate properties.

Table 9-2 Grading Limits, Specified Size and Maximum Flakiness Index for Surface Treatment Aggregates for Lightly Trafficked Roads

Test Sieve (mm)	**Nominal Size of Aggregates (mm)**			
	20	**14**	**10**	**6.3**
28	100	-	-	-
20	85-100	100	-	-
14	0-40	85-100	100	-
10	0-7	0-40	85-100	100
6.3	-	0-7	0-35	85-100
5.0	-	-	0-10	-
3.35	-	-	-	0-35
2.36	0-3	0-3	0-3	0-10
0.600	0-2	0-2	0-2	0-2
0.075	-	-	-	-
	Minimum Percentage by Mass Retained On Test Sieve			
Specified Size	60	60	65	65
Maximum Flakiness Index	35	35	35	-

The nominal sizes of chippings normally used for surface treatment are 6, 10, 14 and 20 mm. Flaky chippings are those with a thickness (smallest dimension) less than 0.6 of their nominal size. The proportion of flaky chippings clearly affects the average thickness of a single layer of the chippings, and it is for this reason that the concept of the ‘average least dimension’ (ALD) of chippings was introduced.

In effect, the ALD is the average thickness of a single layer of chippings when they have bedded down into their final interlocked positions. The amount of binder required to retain a layer of chippings is thus related to the ALD of the chippings rather than to their nominal size. This is discussed further in Section 9.5 where guidance is given on the selection of the appropriate nominal size of chipping and the effect of flakiness on surface treatment design.

The most critical period for a surface treatment occurs immediately after the chippings have been spread on the binder film. At this stage the chippings have yet to become an interlocking mosaic and are held in place solely by the adhesion of the binder film. Dusty chippings can seriously impede adhesion and can cause immediate failure of the dressing.

The effect of dust can sometimes be mitigated by dampening them prior to spreading them on the road. The chippings dry out quickly in contact with the binder and when a cutback bitumen or emulsion is used, good adhesion develops more rapidly than when the coating of dust is dry.

Most aggregates have a preferential attraction for water rather than for bitumen. Hence if heavy rain occurs within the first few hours when adhesion has not fully developed, loss of chippings under the action of traffic is possible. Where wet weather damage is considered to be a severe risk, or the immersion tray test (Appendix A) shows that the chippings have poor affinity with bitumen, an adhesion agent should be used. An adhesion agent can be added to the binder or used in a dilute solution to pre-coat the chippings. However, the additional cost of the adhesion agent will be wasted if proper care and attention is not given to all other aspects of the surface treatment process.

Improved adhesion of chippings to the binder film can also be obtained by pre-treating the chippings before spreading. This is likely to be most beneficial if the available chippings are very dusty or poorly shaped, or if traffic conditions are severe. There are basically two ways of pre-treating chippings:

Spraying the chippings with a light application of creosote, diesel oil, or kerosene at ambient temperature. This can be conveniently done as the chippings are transferred from stockpile to gritting lorries by a belt conveyor or, alternatively, they can be mixed in a simple concrete mixer.

- Pre-coating the chippings with a thin coating of hard bitumen such that the chippings do not stick together and can flow freely.

Chippings which are pre-coated with bitumen enable the use of a harder grade of binder for construction which can provide early strong adhesion and thus help to obtain high quality dressings. The binder used for pre-coating need not necessarily be the same kind as used for the surface treatment. Pre-coating is usually undertaken in a hot-mix plant and the hardness of the coating, and thus the tendency for the chippings to adhere to each other can be controlled by the mixing temperature and/or the duration of mixing; typical coating temperatures are about 140^{o}C for bitumen binders. Table 9.3 indicates the amount of binder recommended for lightly coating chippings.

Pre-coated chippings should not be used with emulsions because the breaking of the emulsion will be adversely affected.

Table 9-3 Binder Contents for Lightly-Coated Chippings

Nominal Size of Chippings (mm)	Target Binder Content (per cent by mass)
6	1.0
10	0.8
14	0.6
20	0.5

Adhesion agents or pre-treatment chippings are often used in an attempt to counteract the adverse effect of some fundamental fault in the surface treatment operation. If loss of chippings has occurred, it is advisable to check whether the viscosity of the binder was appropriate for the ambient road temperature at the time to spraying. The effectiveness of the chipping and traffic control operations should also be reviewed before the use of an adhesion agent or pre-treated chippings is considered.

9.4 Bitumens

It is essential that good bonding is achieved between the surface treatment and the existing road surface. This means that non-bituminous materials must be primed before surface treatment is carried out.

9.4.1 Prime Coats

Where a surface treatment is to be applied to a previously untreated road surface it is essential that the surface should be dry, clean and as dust-free as possible. On granular, cement or lime-stabilised surfaces a prime coat of bitumen ensures that these conditions are met. The functions of a prime coat can be summarized as follows:

i) It assists in promoting and maintaining adhesion between the roadbase and a surface treatment by pre-coating the roadbase and penetrating surface voids.
ii) It helps to seal the surface pores in the roadbase thus reducing the absorption of the first spray of binder of the surface treatment.
iii) It helps to strengthen the roadbase near its surface by binding the finer particles of aggregate together.
iv) If the application of the surface treatment is delayed for some reason it provides the roadbase with a temporary protection against rainfall and light traffic until the surfacing can be laid.

The depth of penetration of the prime should be between 3-10mm and the quantity sprayed should be such that the surface is dry within a few hours. The correct viscosity and application rate are dependent primarily on the texture and density of the surface being primed. The application rate is, however, likely to lie within the range 0.3-1.1 kg/m^2. Low viscosity cutbacks are necessary for dense cement or lime-stabilised surfaces, and higher viscosity cutbacks for untreated coarse-textured surfaces. It is usually beneficial to spray the surface lightly with water before applying the prime coat as this helps to suppress dust and allows the primer to spread more easily over the surface and to penetrate. Bitumen emulsions are not suitable for priming as they tend to form a skin on the surface.

Low viscosity, medium curing cutback bitumens such as MC-30, MC-70, or in rare circumstances MC-250, can be used for prime coats. The relationship between grade and viscosity (see Appendix A) for cutback primes is shown in Table 9.4.

Table 9-4 Kinematic Viscosities of Current Cutback Binders

Grade of Cutback Binder	Permitted Viscosity Range (Centistokes at 60°C)
MC 250	250-500
MC 70	70-140
MC 30	30-60

9.4.2 Bitumens for Surface Treatments

The correct choice of bitumen for surface treatment work is critical. The bitumen must fulfil a number of important requirements. It must:

i) Be capable of being sprayed;

ii) 'Wet' the surface of the road in a continuous film;

iii) Not run off a cambered road or form pools of binder in local depressions;

iv) 'Wet' and adhere to the chipping at road temperature;

v) Be strong enough to resist traffic forces and hold the chippings at the highest prevailing ambient temperatures;

vi) Remain flexible at the lowest ambient temperature, neither cracking nor becoming brittle enough to allow traffic to 'pick-off' the chippings; and

vii) Resist premature weathering and hardening.

Some of these requirements conflict, hence the optimum choice of binder involves a careful compromise. For example, the binder must be sufficiently fluid at road temperature to 'wet' the chippings whilst being sufficiently viscous to retain the chippings against the dislodging effect of vehicle tyres when traffic is first allowed to run on the new dressing.

Figure 9.2 shows the permissible range of binder viscosity for successful surface treatment at various road surface temperatures. In Ethiopia, daytime road temperatures typically lie between about $25^{o}C$ and $50^{o}C$, normally being in the upper half of this range unless heavy rain is falling. For these temperatures the viscosity of the binder should lie between approximately 10^4 and $7x10^5$ centistokes. At the lower road temperatures cutback grades of bitumen are most appropriate, whilst at higher road temperatures penetration grade bitumens must be used.

The temperature/viscosity relationships shown in Figure 9.2 do not apply to bitumen emulsions. These have a relatively low viscosity and 'wet' the chippings readily. After this the emulsion 'breaks,' the water evaporates and particles of high viscosity bitumen adhere to the chippings and the road surface.

Depending upon availability and local conditions at the time of construction, the following types of bitumen are commonly used:

i) Penetration grade

ii) Emulsion

iii) Cutback

iv) Modified bitumens

9.4.3 Penetration Grade Bitumens

Penetration grade bitumens vary between 80/100 to approximately 700 penetration. The softer penetration grade binders are usually produced at the refinery but can be made in the field by blending appropriate amounts of kerosene, diesel, or a blend of kerosene and diesel. With higher solvent contents the binder has too low a viscosity to be classed as being of penetration grade and is then referred to as cutback bitumen which, for surface treatment work, is usually an MC or RC 3000 grade. In very rare circumstances a less viscous grade such as MC or RC 800 may be used if the pavement temperature is below $15^{o}C$ for long periods of the year.

9.4.4 Bitumen Emulsion

Cationic bitumen emulsion with a bitumen content of 70 to 75 per cent is recommended for most surface treatment work. This type of binder can be applied through whirling spray jets at a temperature between 70 and 85°C. Once applied, it will break rapidly on contact with chippings of most mineral types. The cationic emulsifier is normally an anti-stripping agent and this ensures good initial bonding between chippings and the bitumen.

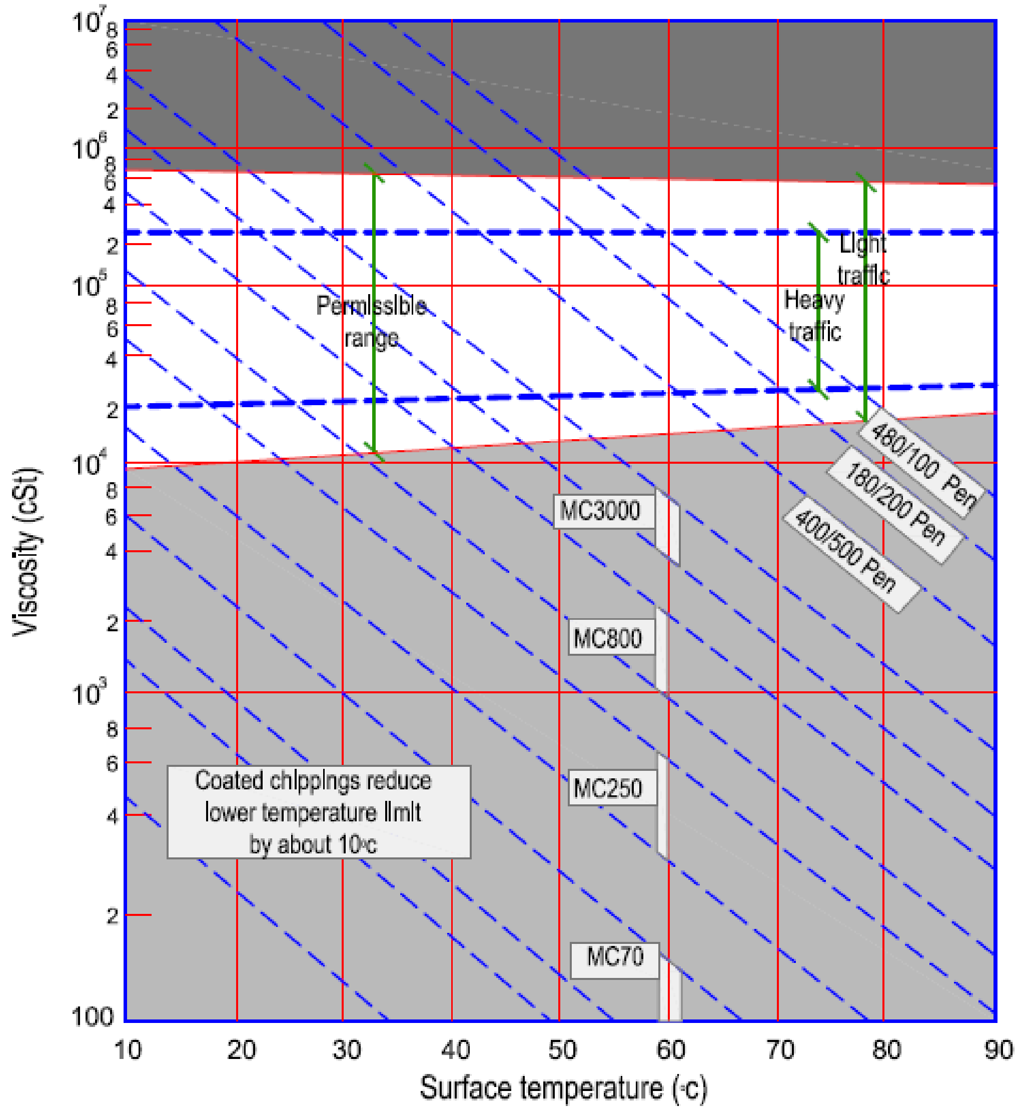


Figure 9-2 Surface Temperature/Choice of Binder for Surface Treatments

When high rates of spray are required, the road is on a gradient, or has considerable camber, the emulsion is likely to drain from the road or from high parts of the road surface before 'break' occurs. In these cases it may be possible to obtain a satisfactory result if the bitumen application is 'split', with a reduced initial rate of spray and a heavier application after the chippings have been applied. If the intention was to construct a single seal then the second application of binder will have to be covered with sand or quarry fines to prevent the binder adhering to roller and vehicle wheels. If a double dressing is being constructed then it should be possible to apply sufficient binder in the second spray to give the required total rate of spray for the finished dressing.

If split application of the binder is used care must be taken with the following:

i) The rate of application of chippings must be correct so that there is a minimum of excess chippings.

ii) The second application of binder must be applied before traffic is allowed onto the dressing.

iii) For a single seal it will be necessary to apply grit or sand after the second application of binder.

9.4.5 Cutback Bitumens

Except for very cold conditions, MC or RC 3000 grade cutback is normally the most fluid binder used for surface treatments. This grade of cutback is basically an 80/100 penetration grade bitumen which has been blended with approximately 12 to 17 percent of cutter.

In Ethiopia, the range of binders available to the engineer may be restricted. In this situation it may then be necessary to blend two grades together or to 'cut-back' a supplied grade with diesel oil or kerosene in order to obtain a binder with the required viscosity characteristics. Diesel oil is preferable to kerosene for blending purposes because it is less volatile than kerosene and is generally more readily available. Only relatively small amounts of diesel oil or kerosene are required to modify penetration grade bitumen such that its viscosity is suitable for surface treatment at road temperatures in Ethiopia. For example, Figure 9.3 shows that between 2 and 10 per cent of diesel oil was required to modify an 80/100 pen bitumen to produce binders with viscosities within the range of road temperatures of between 40°– 60°, which prevail in Ethiopia (Figure 9.2). Figure 9.4 shows the temperature/viscosity relationships for five of the blends made for trials.

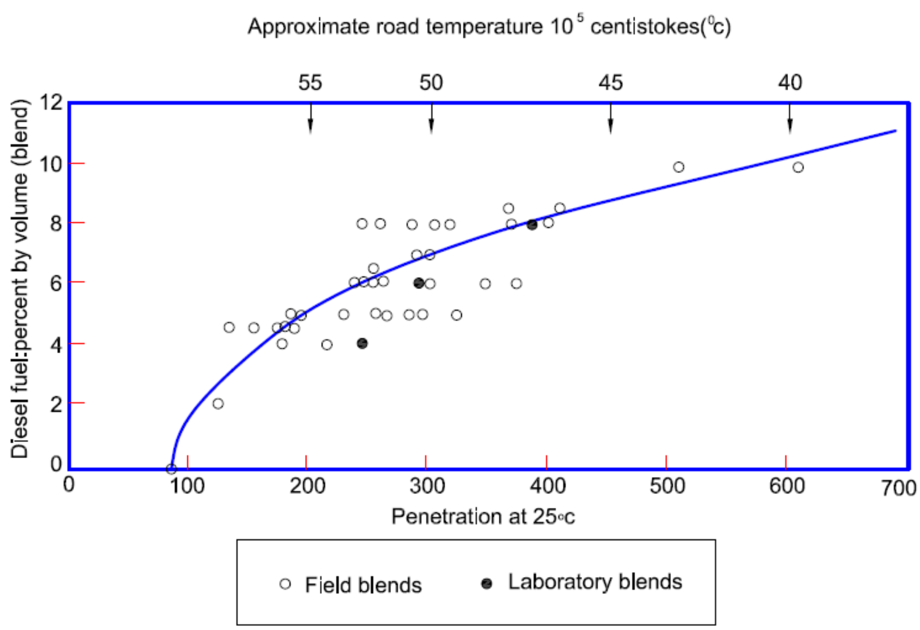


Figure 9-3 Blending Characteristics of 80/100 Pen Bitumen with Diesel Fuel

The amount of cutter required for 'on-site' blending should be determined in the laboratory by making viscosity tests on a range of blends of bitumen and cutter. MC 3000 can be made in the field by blending 90 penetration bitumen with 12 to 14 per cent by volume of a 3:1 mixture of kerosene and diesel. If there is significantly more than 14 per cent of cutter by volume then the spray rate should be adjusted to compensate for this (see Section 9.5.8). For binders that have been cutback at the refinery, the cutter content should be obtained from the manufacturer.

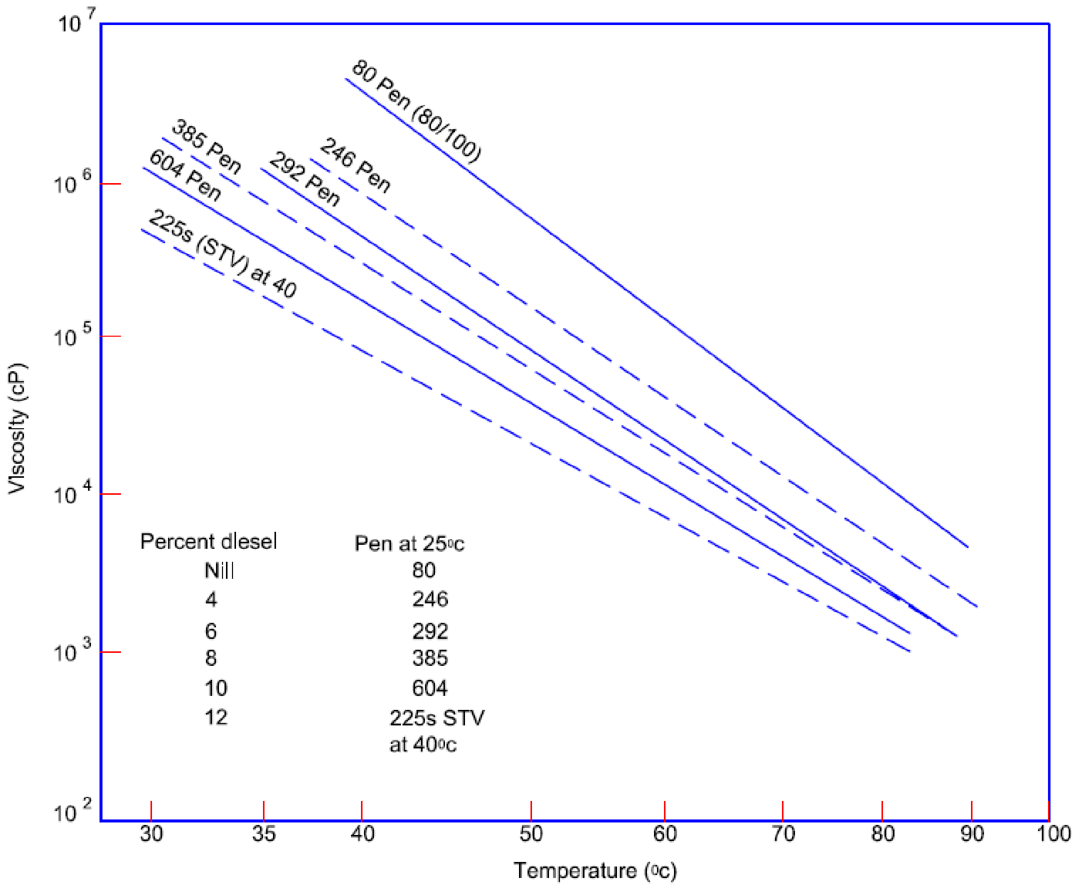


Figure 9-4 Viscosity/Temperature Relationships for Blends of 80/100 Pen Bitumen with Diesel Fuel

9.4.6 Polymer Modified Bitumens

Polymers can be used in surface treatment to modify penetration grade, cutback bitumens and emulsions. Usually these modified binders are used at locations where the road geometry, traffic characteristics or the environment dictate that the road surface experiences high stresses. Generally the purpose of the polymers is to reduce binder temperature susceptibility so that variation in viscosity over the ambient temperature range is as small as possible. Polymers can also improve the cohesive strength of the binder so that it is more able to retain chippings when under stress from the action of traffic. They also improve the early adhesive qualities of the binder allowing the road to be re-opened to traffic earlier than may be the case with conventional unmodified binders. Other advantages claimed for modified binders are improved elasticity in bridging hairline cracks and overall improved durability.

Examples of polymers that may be used to modify bitumens are proprietary thermoplastic rubbers such as Styrene-Butadiene-Styrene (SBS), crumb rubber derived from waste car tyres and also glove rubber from domestic gloves. Latex rubber may also be used to modify emulsions. Binders of this type are best applied by distributors fitted with slotted jets of a suitable size.

Rubber modified bitumen may typically consist of a blend of 80/100 penetration grade bitumen and three per cent powdered rubber. Blending and digestion of the rubber with the penetration grade bitumen should be carried out prior to loading into a distributor. This must be done in static tanks which incorporate integral motor driven paddles. The blending temperature is approximately 200°C.

Cationic emulsion can be modified in specialised plant by the addition of three per cent latex rubber. One of the advantages of using emulsions is that they can be sprayed at much lower temperatures than penetration grade bitumens. This reduces the risk of partial degradation of the rubber which can occur at high spraying temperatures.

Bitumen modified with SBS exhibits thermoplastic qualities at high temperatures while having a rubbery nature at lower ambient temperatures. With three per cent of SBS, noticeable changes in binder viscosity and temperature susceptibility occur and good early adhesion of the chippings is achieved. SBS can be obtained in a carrier bitumen in blocks of approximately 20kg mass. The blocks can be blended, at a concentration recommended by the manufacturer, with 80/100 penetration binder in a distributor. In this procedure it is best to place half of the required polymer into the empty distributor, add hot bitumen from a main storage tank and then circulate the binder in the distributor tank. The remaining blocks are added after about 30 minutes and then about 2 hours is likely to be required to complete blending and heating of the modified binder. Every effort should be made to use the modified bitumen on the day it is blended.

9.4.7 Adhesion Agents

Fresh hydrated lime can be used to enhance adhesion. It can be mixed with the binder in the distributor before spraying (slotted jets are probably best suited for this) or the chippings can be pre-coated with the lime just before use by spraying with lime slurry. The amount of lime to be blended with the bitumen should be determined in laboratory trials but approximately 12 per cent by mass of the bitumen will improve bitumen-aggregate adhesion and it should also improve the resistance of the bitumen to oxidative hardening.

Proprietary additives, known as adhesion agents, are also available for adding to binders to help to minimize the damage to surface treatments that may occur in wet weather with some types of stone. When correctly used in the right proportions, these agents can enhance adhesion between the binder film and the chippings even though they may be wet. The effectiveness and the amount of an additive needed to provide satisfactory adhesion of the binder to the chippings in the presence of free water must be determined by tests such as the Immersion Tray Test (Appendix A).

Cationic emulsions inherently contain an adhesion agent and lime should not be used with this type of binder.

9.5 Design

The key stages in the surface treatment design procedure are illustrated in Figure 9.5.

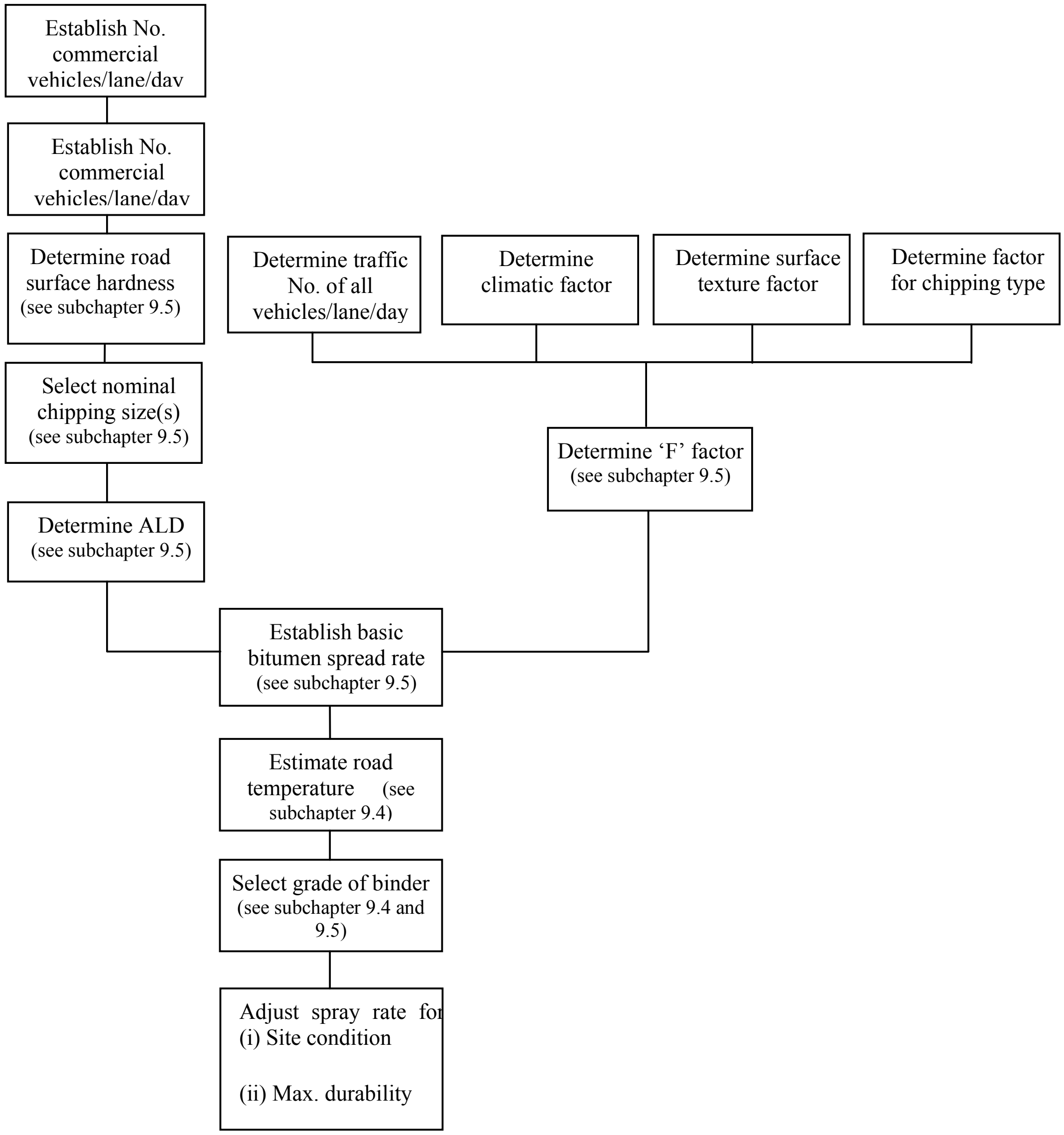


Figure 9-5 Outline Procedure for Design of Surface Treatments

9.5.1 Existing Site Conditions

Selection of a suitable surface treatment system for a road and the *nominal* size of chippings to be used are based on the daily volume of commercial vehicles using each lane of the road and the hardness of the existing pavement surface.

With time, the action of traffic on a surface treatment gradually forces the chippings into the underlying surface, thus decreasing the surface texture. When the loss of surface texture reaches an unacceptable level, a reseal will be required to restore skid resistance. The embedment process occurs more rapidly when the underling road surface is softer, or when the volume of traffic, particularly of commercial vehicles, is high. Accordingly, larger chippings are required on soft surfaces or where traffic is heavy whilst small chippings are best for hard surfaces. For example, on a very soft surface carrying 1000 commercial vehicles per lane per day, 20mm chipping are appropriate, whilst on a very hard surface such as concrete, 6mm chipping should be the best choice.

Guidance on the selection of chipping size for single surface treatments, relating the nominal size of chipping to the hardness of the underlying road surface and the weight of traffic expressed in terms of the number of commercial vehicles carried per lane per day, is shown in Table 9.5.

Table 9-5 Recommended Nominal Size of Chippings (mm)

Type of Surface	Approximate number of commercial vehicles with an unladen weight greater than 1.5 tonnes currently carried in the design lane				
	2000-4000	1000-2000	200-1000	20-200	Less than 20
Very Hard	10	10	6	6	6
Hard	14	14	10	6	6
Normal	20[1]	14	10	10	6
Soft	*	20[1]	14	14	10
Very Soft	*	*	20[1]	14	10

Notes 1 The size of chipping specified is related to the mid-point of each lane traffic category. Lighter traffic conditions may make the next smaller size of stone more appropriate.

2 Very particular care should be taken when using 20mm chippings to ensure that no loose chippings remain on the surface when the road is opened to unrestricted traffic as there is a high risk of windscreen breakage.

3 * Unsuitable for surface treatment.

Road surface hardness may be assessed by a simple penetration probe test (Appendix J). This test utilizes a modified soil assessment cone penetrometer. Alternatively the hardness of the existing road surface may be made on the basis of judgement with the help of the definitions given in Table 9.6.

Table 9-6 Categories of Road Surface Hardness

Category of Surface	Penetration at 30°C[1]	Definition
Very Hard	0 - 2	Concrete or very lean bituminous structures with dry stony surfaces. There would be negligible penetration of chippings under the heaviest traffic
Hard	2 - 5	Likely to be an asphalt surfacing which has aged for several years and is showing some cracking. Chippings will penetrate only slightly under heavy traffic
Normal	5 - 8	Typically, an existing surface treatment which has aged but retains a dark and slightly bitumen-rich appearance. Chippings will penetrate moderately under medium and heavy traffic.
Soft	8 - 12	New asphalt surfacing or surface treatments which look bitumen-rich and have only slight surface texture. Surfaces into which chippings will penetrate considerable under medium and heavy traffic.
Very soft	>12	Surfaces, usually a surface treatment which is very rich in binder and has virtually no surface texture. Even large chippings will be submerged under heavy traffic

If larger sized chippings are used than those recommended in Table 9.5 then the necessary bitumen spray rate required to hold the chippings in place is likely to be underestimated by the design procedure described in this subchapter. This is likely to result in the ‘whip-off’ of chippings by traffic early in the life of the dressing and also to have a significant effect on the long term durability of low volume roads.

In selecting the nominal size of chippings for double surface treatments the size of chipping for the first layer should be selected on the basis of the hardness of the existing surface and the traffic category as indicated in Tables 9.5 and 9.6. The nominal size of chipping selected for the second layer should preferably have an ALD of not more than half that of the chippings used in the first layer. This will promote good interlock between the layers.

In the case of a hard existing surface where very little embedment of the first layer of chippings is possible, such as newly constructed cement stabilised road base or a dense crushed rock base, a 'pad coat' of 6mm chippings should be applied first followed by 10mm or14mm chippings in the second layer. The first layer of small chippings will adhere well to the hard surface and will provide a 'key' for the larger stone of the second dressing.

9.5.2 Selecting the Binder

The selection of the appropriate binder for a surface treatment is usually constrained by the range of binders available from suppliers, although it is possible for the user to modify the viscosity of penetration grade and cutback binders to suit local conditions as described in Section 9.4.

The factors to be taken into account in selecting an appropriate binder are:

The road surface temperature at the time the surface treatment is undertaken. For penetration grade and cutback binders the viscosity of the binder should be between 10^4 and $7x10^5$ centistokes at the road surface temperature (see Section 9.4 and Figure 9.2).

i) *The nature of the chippings.* If dusty chippings are anticipated and no pre-treatment is planned, the viscosity of the binder used should be towards the lower end of the permissible range. If the binder selected is an emulsion it should be borne in mind that anionic emulsions may not adhere well to certain acidic aggregates such as granite and quartzite.

ii) *The characteristics of the road site.* Fluid binders such as emulsions are not suited to steep crossfalls or gradients since they may drain off the road before 'breaking'. However, it may be possible to use a 'split application' of binder.

iii) *The type of binder handling and spraying equipment available.* The equipment must be capable of maintaining an adequate quantity of the selected binder at its appropriate spraying temperature and spraying it evenly at the required rate of spread.

iv) The available binders. There may be limited choice of binders but a balanced choice should be made where possible. Factors which may influence the final selection of a binder include cost, ease of use, flexibility with regard to adjusting binder viscosity on site and any influence on the quality of the finished dressing.

Consideration of these factors will usually narrow the choice of binder to one or two options. The final selection will be determined by other factors such as the past experience of the surface treatment team.

9.5.3 Choice of Binder and Timing of Construction Work

The choice of cutback grade or penetration grade bitumen for surface treatment work is largely controlled by road temperatures at and shortly after the time of construction. However, there are relative advantages and disadvantages associated with the use of penetration grade binders or cutback bitumen.

MC 3000 cutback binder typically contains 12 to 17 per cent of cutter. Under warm road conditions this makes the binder very tolerant of short delays in the application of chippings and of the use of moderately dusty chippings. It is therefore a good material to use. However, a substantial percentage of the cutter, especially if it is diesel, can remain in the seal for many months. If road temperatures increase soon after construction it is likely that MC 3000 will be found to be 'tender' and that the seal can be easily damaged. This should not be a problem for lightly trafficked roads and for new roads that are not opened to general traffic for several days after the surface treatment is constructed. If a road must be opened to fast high volume traffic within a few hours of construction then there will be considerable advantage in using as high a viscosity binder as conditions will permit. For example, if the road temperature is 40^{o}C then, for heavy traffic, the chart in Figure 9.2 suggests that MC 3000 would be only just viscous enough. On the other hand 400/500 penetration grade bitumen would be on the limit of being too viscous, however, it would be preferable to cut-back the bitumen to a 500/600 penetration grade rather than use MC 3000. If pre-coated chippings could be used then 400 penetration grade bitumen would be acceptable.

Penetration grade bitumens as hard as 80/100 are often used for surface treatment work when road temperatures are high. With such high viscosity bitumen it is very important that the chippings are applied immediately after spraying and, to achieve this, the chipping spreader must follow closely behind the distributor. This type of binder will not be tolerant of delays in the application of the chippings nor of the use of dusty chippings. In either situation, early trafficking is very likely to dislodge chippings and seriously damage the seal.

The use of penetration grade binders in the range 80/100 to 400 is preferred to MC 3000 wherever circumstances allow this. For high volume fast traffic, where very early adhesion of the chippings is essential, consideration should be given to the use of pre-coated chippings. This will allow the use of a more viscous binder for a given road temperature and will ensure that a strong early bonding of the chipping is obtained. A polymer modified or rubberized binder can also provide immediate strong adhesion. Alternatively, emulsions will provide good 'wetting' and early adhesion provided rainfall does not interfere with curing.

The most difficult situations occur when it is required to start work early in the day and temperatures are considerably lower than they will be in the afternoon. It may appear to be appropriate to user a cutback binder such as MC 3000 for the low road temperature but, by the afternoon, the seal is likely to be too 'soft.' In these situations it is better to use a more viscous binder and keep the traffic off of the new seal until it has been rolled in the afternoon.

9.5.4 Designing the Surface Treatment

Having selected the nominal size of chipping and the type of binder to be used, the next step in the design of a surface treatment is to determine the rate of spread of the binder.

Differences in climate, uniformity of road surfaces, the quality of aggregates, traffic characteristics and construction practice, necessitate a general approach to the determination of the rate of spread of the binder for application in Ethiopia.

The method of design relates the voids in a layer of chippings to the amount of binder necessary to hold the chippings in place. In a loose single layer of chippings such as is spread for a surface treatment, the voids are initially about 50 per cent, decreasing to about 30 per cent after rolling and subsequently to 20 per cent by the action of traffic. For best results, between 50 and 70 per cent of the voids in the compacted aggregate should be filled with binder. Hence it is possible to calculate the amount of binder required to retain a layer of regular, cubical chipping of any size. However, in practice chippings are rarely the ideal cubical shape (especially when unsuitable crushing plant has been used) and this is why the ALD concept was originally introduced.

9.5.5 Determining the Average Least Dimension (ALD) of Chippings

The ALD of chippings is a function of both the average size of the chippings, as determined by normal square mesh sieves, and the degree of flakiness. The ALD may be determined in two ways:

Method A: A grading analysis is performed on a representative sample of the chippings in accordance with ASTM C136. The sieve size through which 50 per cent of the chippings pass is determined (i.e. the 'median size'). The flakiness index is then also then derived from the nomograph shown in Figure 9.6.

Method B: A representative sample of the chipping is carefully subdivided (in accordance with British Standard 812: 1985) to give approximately 200 chippings. The least dimension of each chipping is measured manually and the mean value, or ALD, is calculated.

9.5.6 Determining the Overall Weighting Factor

The ALD of the chippings is used with an overall weighting factor to determine the basic rate of spray of bitumen. The overall weighting factor 'F' is determined by adding together four factors that represent; the level of traffic, the condition of the existing road surface, the climate and the type of chippings that will be used. Factors appropriate to the site to be surface dressed are selected from Table 9.7.

For example, if flaky chippings (factor = –2) are to be used at a road site carrying medium to heavy traffic (factor = –1) and which has a primed base surface (factor = +6) in a wet tropical climate (factor =+1) the overall weighting factor 'F' is:

$$F = -2 - 1 + 6 + 1 = +4$$

The rating for the existing surface allows for the amount of binder that is required to fill the surface voids and which is therefore not available to contribute to the binder film that retains the chippings. If the existing surface of the road is rough, it should be rated as 'very lean bituminous' even if its overall colour is dark with bitumen. Similarly, when determining the rate of spread of binder for the second layer of a double surface treatment, the first layer should also be rated 'very lean bituminous'.

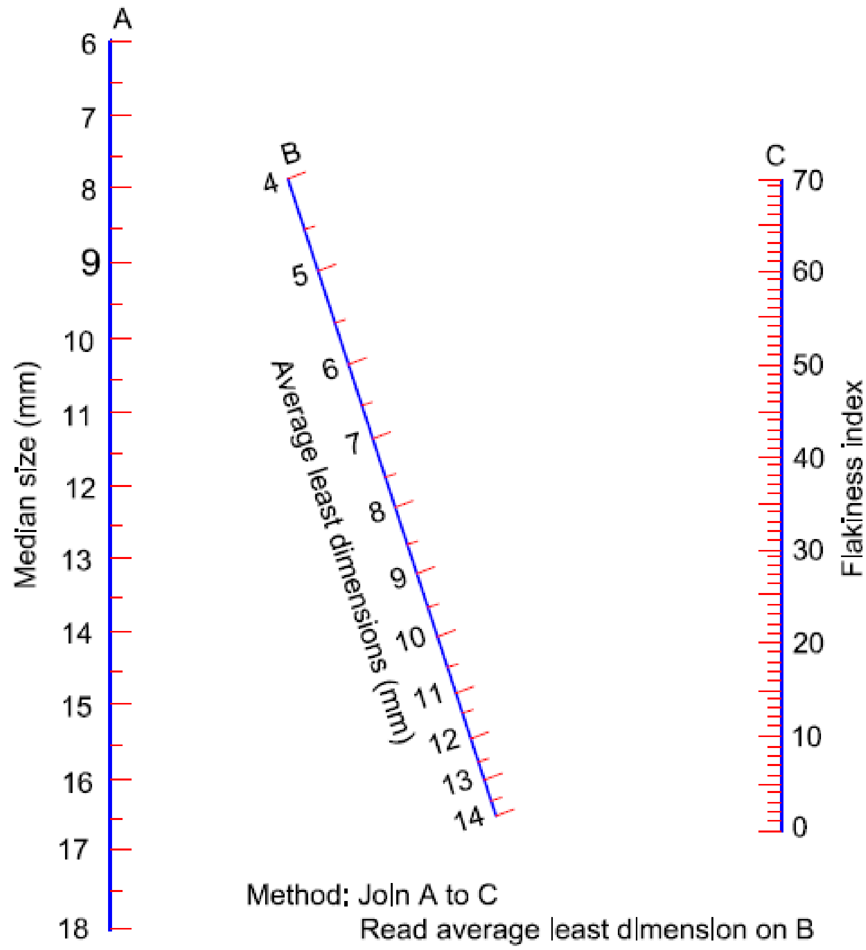


Figure 9-6 Determination of Average Least Dimension

This method of determining the rate of spread of binder requires the estimation of traffic in terms of numbers of vehicles only. However if the proportion of commercial vehicles in the traffic stream is high (say more than 20 per cent) the traffic factor selected should be for the next higher category of traffic than is indicated by the simple volume count.

Table 9-7 Weighting Factors for Surface Treatment Design

Description	Vehicles/lane/day	Factor
Total traffic (all classes)		
Very light	0 - 20	+8
Light	20 – 100	+4
Medium light	100 - 250	+2
Medium	250 - 500	0
Medium – heavy	500 – 1500	-1
Heavy	1500 – 3000	-3
Very heavy	3000 +	-5
Existing surface		
Untreated or primed base		+6
Very lean bituminous		+4
Lean bituminous		0
Average bituminous		-1
Very rich bituminous		-3
Climatic Conditions		
Wet and cold		+2
Tropical (wet and hot)		+1
Temperate		0
Semi arid (hot and dry)		-1
Arid (very dry and very hot		-2
Type of Chippings		
Round/dusty		+2
Cubical		0
Flaky (see Tables 9.1 and 9.2		-2
Pre-coated with bitumen		-2

9.5.7 Determining the Basic Bitumen Spray Rate

Using the ALD and 'F' values in equation 1 will give the required *basic* rate of spread of binder.

$$R = 0.6250 + 0\,(F*0.023) + [0.0375 + (F*0.0011)]\ ALD \qquad (1)$$

Where F = Overall weighting factor

ALD = The average least dimension of the chippings (mm)

R = Basic rate of spread of bitumen (kg/m^2)

Alternatively, the values for F and ALD can be used in the design chart given in Figure 9.7. The intercept between the appropriate factor line and the ALD line is located and the basic rate of spread of the binder is then read off directly at the bottom of the chart. The basic rate of spread of bitumen (R) is the mass of MC 3000 binder per unit area on the road surface immediately after spraying. The relative density of MC 3000 can be assumed to be 1.0 and the spread rate can therefore also be expressed in litres/m^2; however, calibration of a distributor is easier to do by measuring spray rates in terms of mass.

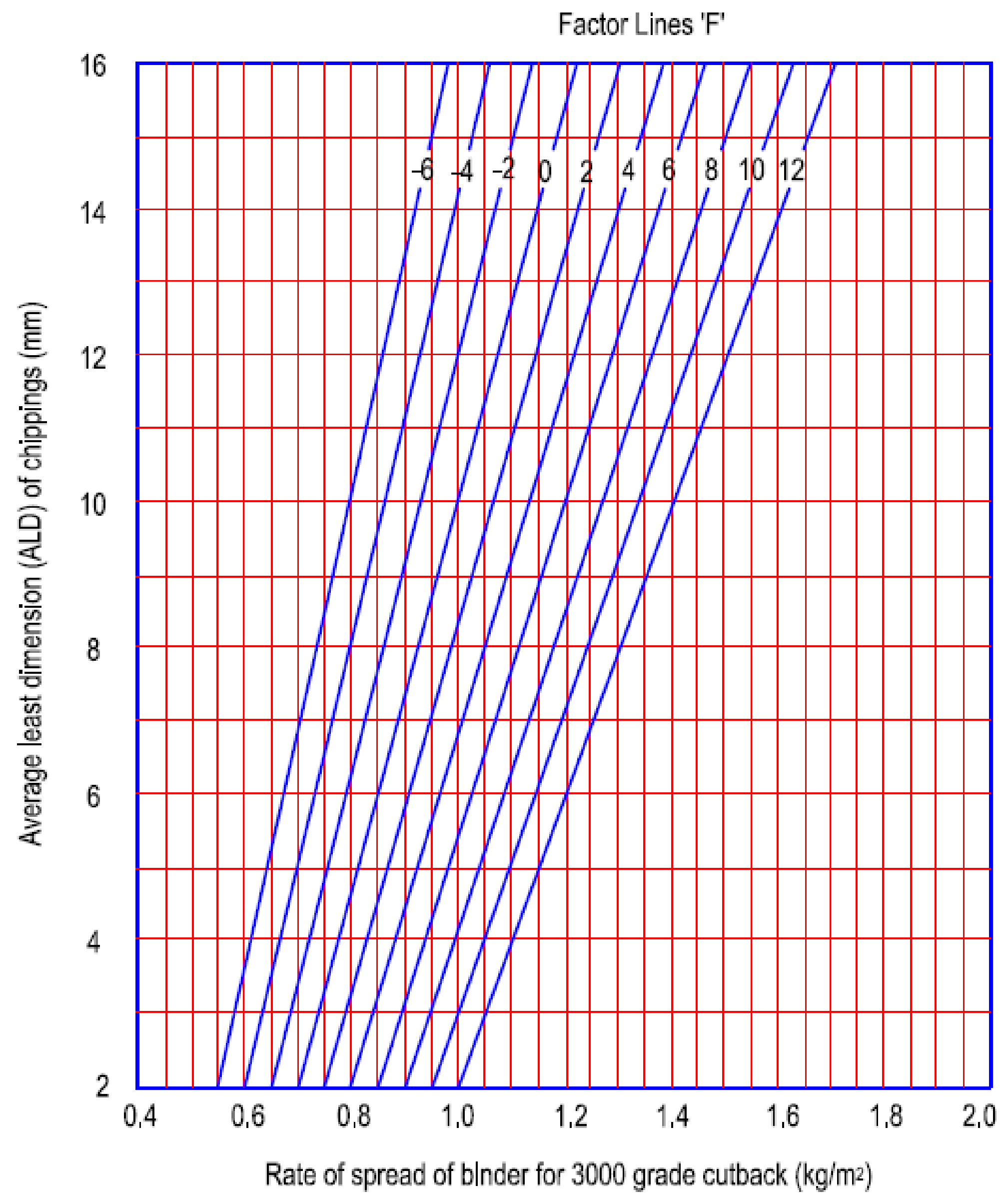


Figure 9-7 Surface Treatment Design Chart

9.5.8 Spray Rate Adjustment Factors

Adjustments to the basic spray rates are required for the following reasons;

i) Type of bitumen (to take account of the difference between the residual binder content of MC3000 on which the designs are based and the binder being used).

ii) Speed of traffic and road gradient.

The basic rate of spread of binder must also be modified to allow for the type of binder used because the rates quoted are for MC 3000. The following modifications are required:

Penetration grade binders.

The rate of spread should be decreased by 10 per cent.

Cutback binders

For MC/RC3000 no modification is required. If a different grade of binder is required then the adjustment factor should reflect the different amount of cutter used. For instance, a 200 penetration binder may have 3 per cent cutter in it compared with 80/100 pen grade and therefore the spray rate is 103 per cent of the rate for an 80/100 pen grade and 1.03 x 0.9 = 0.93 times the rate for MC 3000. In the same way, in the rare cases when cutbacks with lower viscosity are used, the rate of spread should be increased to allow for the additional percentage of cutter.

Emulsions

The binder in the emulsion is equivalent to an 80/100 penetration grade and this requires an adjustment of 0.9 times the MC 3000 spray rate. But there is also considerable volume of water, for example 30%, hence to adjust for this and to arrive at the correct amount of residual bitumen requires an increase in spray rate of a factor of 1.3, hence the total adjustment factor is 1.3 x 0.9 = 1.17

It has also been found that the best results will be obtained if the basic rate of spread of binder is also adjusted to take account of traffic speed and road gradient as follows:

For slow traffic or climbing grades with gradients steeper than 3 per cent, the basic rate of spread of binder should be reduced by 10 per cent.

- For fast traffic or down-grades steeper than 3 per cent the basic rate of spread of binder should be increased by approximately 10 per cent.

The definition of traffic speed is not precise but is meant to differentiate between roads with a high proportion of heavy vehicles and those carrying mainly cars travelling at 80km/h or more.

The adjustment factors for different binders and different site conditions are summarised in Table 9.8.

The spray rate which will be arrived at after applying the adjustment factors in Table 9.8 will provide very good surface texture and use an economic quantity of binder. However, because of the difficulties experienced in carrying out effective maintenance, there is considerable merit in sacrificing some surface texture for increased durability of the seal. For roads on flat terrain and carrying moderate to high-speed traffic it is possible to increase the spray rates obtained from Table 9.8 by approximately 8 per cent. The heavier spray rate may result in the surface having a 'bitumen-rich' appearance in the wheel paths of roads carrying appreciable volumes of traffic. However, the additional binder should not result in bleeding and it can still be expected that more surface texture will be retained than is usual in an asphalt concrete wearing course.

Table 9-8 Typical Bitumen Spray Rate Adjustment Factors

Binder Grade	Basic Spray Rate from Figure 9.7 or Equation 1	Flat Terrain, Moderate Traffic Speed	High Speed Traffic, Down-Hill Grades>3%	Low Speed Traffic, Up-Hill Grades>3%
MC 3000	R	R	R*1.1	R*0.9
300 pen	R	R*0.95	R*1.05	R*0.86
80/100 pen	R	R*0.9	R*0.99	R*0.81
Emulsion[1]	R	R*(90/% binder)	R*(99/% binder)	R*(81/% binder)

9.5.9 Surface Treatment Design for Low Volume Roads

If a low volume road carrying less than about 100 vehicles per day is surface dressed it is very important that the seal is designed to be as durable as possible to minimize the need for subsequent maintenance.

A double surface treatment should be used on new road bases and the maximum durability of the seal can be obtained by using the heaviest application of bitumen that does not result in bleeding.

On very low volume roads the chippings are unlikely to be 'rotated' by traffic into a matrix as tight as is normally obtained before the bitumen becomes too stiff for the stones to be able to move. This results in the eventual stable layer of chippings being of slightly greater depth than their ALD (which is assumed in the design process). Furthermore, a high level of texture depth to provide good skid resistance is less important on very low volume roads. It is therefore possible to make use of the increased texture depth to enhance durability by increasing the bitumen spray rates. The 'factors' for very low traffic levels in Table 9.8 are designed to accomplish this. Together with the additional 8% of binder discussed in Section 9.5.8 above, very durable surfaces can be designed. However, there is considerable variability in the ideal spray rate for such roads and therefore it is important that these increased spray rates are adjusted on the basis of trial sections and local experience.

Where crushing facilities are put in place solely to produce chippings for a project, it is important to maximize use of the crusher output. This requires the use of different combinations of chipping sizes and correspondingly different bitumen spray rates.

The normally recommended sizes of chippings for different road hardness and low commercial traffic volumes are given in Table 9.9 but it may sometimes be desirable to use chippings of a larger size for reasons of economy.

Ideally the ALD of the two aggregate sizes used in a double surface treatment should differ by at least a factor of two. If the ALD of the chippings in the second seal is more than half the ALD of the chippings in the first seal then the texture depth will be further increased and the capacity of the aggregate structure for bitumen will be increased.

Table 9-9 Nominal Size of Chippings for Different Hardness of Road Surface

No. of commercial vehicles/lane/day[1]	20 - 100	Less than 20
Road Surface Hardness	**Nominal Chipping Size (mm)**	
Very hard	6	6
Hard	6	6
Normal	10	6
Soft	14	10

Note 1 Vehicles with an unladen weight greater than 1.5 tonnes

9.5.10 Spread Rate of Chippings

An estimate of the rate of application of the chippings, assuming that the chippings have a loose density of $1.35 Mg/m^3$, can be obtained from the following equation:

$$\text{Chipping application rate } (kg/m^2) = 1.364 * ALD \qquad (2)$$

The chipping application rate should be regarded as a rough guide only. It is useful in estimating the quantity of chippings that is required for a surface treatment project before crushing and stockpiling of the chippings is carried out. A better method of estimating the approximate application rate of the chippings is to spread a single layer of chippings taken from the stockpile on a tray of known area. The chippings are then weighed, the process repeated ten times with fresh chippings, and the mean value calculated. An additional ten per cent is allowed for whip off. Storage and handling losses must also be allowed for when stockpiling chippings.

The precise chipping application rate must be determined by observing on site whether any exposed binder remains after spreading the chippings, indicating too low a rate of application of chippings, or whether chippings are resting on top of each other, indicating too high an application rate. Best results are obtained when the chippings are tightly packed together, one layer thick. To achieve this, a slight excess of chippings must be applied. Some will be moved by the traffic and will tend to fill small areas where there are insufficient chippings. Too great an excess of chippings will increase the risk of whip-off and windscreen damage.

9.6 Example of a Surface Treatment Design

Site Description

A two-lane trunk road at an altitude of approximately 1500m.

Vehicle count averaged 3370 per day/lane (i.e. ‘Heavy’ rating).

The bitumen to be used is 400 penetration grade (made by cutting back 80/100 pen bitumen with 6.7 per cent by mass or approximately 7.5 per cent by volume) of a 3:1 mixture of kerosene and diesel.

Table 9-10 Design Factors (Example)

Design		Factor
Traffic	Heavy	-3
Existing surface	Average bituminous	-1
Chippings	Cubical	0
Climate	Hot/dry	-1
Overall weighting factor		-5

Aggregate

Nominal size	19 mm
Medium Size (i.e. 50 per cent passing)	16 mm
Flakiness Index	16
Average Least Dimension (from nomograph, Figure 9-6)	12

The determination of spread rates of 80/100 and 400 pen bitumen for an F factor of –5 and an ALD of 12 on a site where maximum durability is required are summarized in Table 9.11.

Table 9-11 Determination of Spread Rates for 400 Penetration Grade Bitumen

Type of terrain	Basic Spread Rate R for MC 3000 (from Fig. 9.7 or Equation 1) (kg/m^2)	For Increased Durability R_D = (R* 1.08) (kg/m_2)	Spread Rates for Penetration Grade Binders (kg/m^2)	
			80/100 pen (R_D*0.9)	400 pen (R_D*0.9*1.067)
Flat	0.89	0.96	0.87	0.92
Uphill Grade > 3%	0.89*0.9 = 0.80	0.87	0.78	0.84
Downhill Grade > 3%	0.89*1.1 = 0.98	1.06	0.95	1.02

Notes
1. For slow traffic or climbing grades steeper than 3 per cent, reduce the rate of spread of binder by 10 per cent.
2. For fast traffic or downgrades steeper than 3 per cent, increase the rate of spread of binder by 10.

9.7 Otta Seals

An Otta seal differs from a surface treatment in that a graded gravel or crushed aggregate containing all sizes including filler is used instead of single-sized chippings. There is no fully comprehensive design procedure but recommendations based on case studies have been published by the Norwegian Public Roads Administration (1999) and considerable experience in their use in Africa has developed during the last 20 years or more. Otta seals may be applied in a single or double layer. They can easily carry 300 vehicles per day and, if constructed well, they can carry considerably more and are comparable with double surface treatments.

The requirements for the aggregate cover material are less stringent than for DBST chippings. The grading of the material is based on the level of traffic expected. Recommended grading envelopes are shown in Table 9.11 and other properties in Table

9.12. Generally, for roads carrying light traffic (<100 vehicles per day), a 'coarse' or 'open' grading should be chosen while a denser grading should be applied on roads carrying higher traffic levels. An open grading should not be used for traffic greater than 1000 vpd.

Table 9-12 Otta Seal Aggregate Grading Requirements

AASHTO Sieve (mm)	Open Grading AADT <100	Medium Grading AADT :100 – 1,000	Dense Grading AADT>1000
	% passing	% passing	% passing
19	100	100	100
16	80 - 100	84 - 100	93 - 100
13.2	52 - 82	68 - 94	84 - 100
9.5	36 - 58	44 - 73	70 - 98
6.7	20 - 40	29 - 54	54 - 80
4.7	10 - 30	19 - 42	44 - 77
2.0	0 - 8	3 - 18	20 - 48
1.18	0 - 5	1 - 14	15 - 38
0.425	0 - 2	0 - 6	7 - 25
0.075	0 - 1	0 - 2	3 - 10

Table 9-13 Properties of Suitable Aggregates for Otta Seals

Property	Requirement
Los Angeles Abrasion	< 35%
Dry 10 % FACT [kN] (min)	180 (T > 3.0 mesa) 150 (0.7 < T < 3.0 mesa) 130 (T < 0.7 mesa)
Sodium Sulphate Soundness	< 10%
Magnesium Sulphate Soundness	< 12%
Flakiness Index	< 30%
Wet and dry strength ratio	> 60%
Water Absorption	< 1%

Aggregates shall be screened natural gravel, crushed rock, crushed stone or crushed gravel of uniform quality. They must be hard, durable, and rounded or cubical in shape. Some cut faces in gravel material are beneficial. As-dug gravels should be screened to remove oversized material and excessive fines (the amount of fines (<0.075mm) must not exceed 10%). Aggregates must be free from harmful, deleterious or objectionable material such as, organic matter, and from excess of flat and elongated pieces, and be dry. Aggregates shall be of such a nature that when thoroughly coated with bitumen material proposed for the work, the coating will not be removed upon contact with water

The preferred maximum particle size is 16mm, but 19mm can be accepted in the first seal where a double seal is to be constructed.

The rate of application of aggregate shall be as specified by the Engineer following the construction of the trial lengths. It is likely to be in the range 0.013 to 0.016 m^3/m^2.

The binders used for Otta seals are normally cut back bitumen MC 800, MC 3000 or 150/200 penetration grade bitumen. The viscosity of the binder selected depends on the ambient temperature and the quality of aggregate employed. 150/200 penetration grade bitumen or MC 3000 cutback grade bitumen is used in warm weather. In cold weather, when night temperatures are likely to fall below 10^{o}C, MC 800 cutback grade bitumen may be used or, alternatively, 150/200 penetration grade bitumen cutback with power paraffin to the appropriate viscosity range as directed by the Engineer

Spray rates cannot be calculated by design and must be chosen empirically. Typically, spray rates (hot) for single seals are between 1.6 and 2.0 $1/m^2$. The nature of the aggregate and the spray rate will determine the amount of bitumen available to bind the aggregate and therefore the eventual thickness of the seal. Excess aggregate will be wasted or, if insufficient aggregate is applied, the seal will be too rich in bitumen. Both scenarios are unsatisfactory and should be avoided. It is because of the broad range of materials that may be used and the empirical nature of the design of this type of seal that it is imperative that pre-construction trials be carried out. This strategy will allow any special local conditions concerning the available aggregates and binders to become apparent to enable the engineer to adjust the nominal design.

An important aspect of Otta seal construction is the need for extensive rolling by pneumatic rollers for at least two days after construction. The action of rolling ensures that the binder is forced upwards, coating the aggregate and thereby initiating the process, that is continued by subsequent trafficking, of forming a ‘premix-like’ appearance to the surface. Rolling should proceed immediately after spreading the aggregate. The first layer should receive not less than 20 passes of a roller, preferably pneumatic or loaded trucks. During the following two days, the entire sealed area, including the shoulders, should receive a further minimum of 15-20 passes daily unless otherwise approved by the Engineer.

After-care can take as long as twelve days and involves sweeping dislodged aggregate back into the wheel paths for further compaction by traffic.

9.8 Other Surface Treatments

Additional information concerning alternative surface treatments, particularly for low volume roads, can be found in ERAs *Best Practice Manual for Thin Bituminous Surfacings* published in 2013 and in the *Low Volume Roads Design Manual*.

9.8.1 Slurry Seals and Cape Seals

A slurry seal is a mixture of fine aggregates, Portland cement filler, bitumen emulsion and additional water (ASTM D 3910). When freshly mixed they have a thick creamy consistency and can be spread to a thickness of 5 to 10 mm. This method of surfacing on its own is not normally used for new construction because it is more expensive than other surface treatments, it does not provide as good a surface texture, and it is not as durable as other properly designed and constructed surface treatments.

Slurry seals are often used in combination with a surface treatment to make a ‘Cape-seal’. In this technique the slurry seal is applied on top of a single surface treatment to produce a surface texture which is less harsh than a surface treatment alone and a surface which is flexible and durable. However, the combination is more expensive than a double surface treatment and requires careful control during construction.

Both anionic and cationic emulsions may be used in slurry seals but cationic emulsion is normally used in slurries containing acidic aggregates, and its early breaking characteristics are also advantageous when rainfall is likely to occur. Suitable specifications for slurry seals and for a Cape-seal are given in Tables 9.12 and 9.13. The optimum mix design for the aggregate, filler, water and emulsion mixture should be determined using ASTM D 3910-84 (1996).

Table 9-14 Aggregate Particle Size Distribution for Slurry Seals

BS Test Sieve (mm)	Percentage by Mass of Total Aggregate Passing Test Sieve		
	Fine	General	Coarse
10	-	100	100
5.0	100	90-100	70-90
2.36	90-100	65-90	45-70
1.18	65-90	45-70	28-50
0.6	40-60	30-50	19-34
0.3	25-42	18-30	12-25
0.15	15-30	10-21	7-18
0.075	10-20	5-15	5-15
Bitumen content (per cent by mass of dry aggregate)	10-16	7.5 - 13.5	6.5 - 12.0

Table 9-15 Typical Coverage for a New 'Cape Seal'

Size of chipping (mm)	Coverage (m^2/m^3)
20	130-170
14	170-240
10	180-250

9.8.2 Sand Seals

Where chippings for a surface treatment are unobtainable or are very costly, sand can be used as 'cover material' for a seal. Sand seals are much less durable than surface treatments; the surface tends to abrade away under traffic. Nevertheless a sand seal can provide a satisfactory surfacing for lightly trafficked roads carrying less than 100 vehicles per lane per day.

It is not possible to design a sand seal in the same sense that a surface treatment can be designed. The particles of sand become submerged in the binder film, and the net result is a thin layer of sand-binder mixture adhering to the road surface.

The sand should be clean and coarse, with a maximum size of 6mm, containing no more than 15 per cent of material finer than 0.3mm and a maximum of 2 per cent of material finer than 0.15mm. The sand should be applied at a rate of 6 to 7×10^{-3} m^3/m^2 The binder, which may be a cutback or an emulsion, should be spread at a rate of approximately 1.0 to 1.2 kg/m^2 depending on the type of surface being sealed.

9.8.3 Synthetic Aggregate and Resin Treatments

These treatments are costly and are used only on relatively small areas, usually in urban situations where high skidding resistance is required. The aggregate is normally a small, single-sized, calcined bauxite which has a high resistance to polishing under traffic. The aggregate is held by a film of epoxy-resin binder. The process requires special mixing and laying equipment and is normally undertaken by specialist contractors.

9.8.4 Applications of Light Bitumen Sprays

A light film of binder can be applied as the final spray on a new surface treatment. The advantage of this procedure is that the risk of whip-off of chippings under fast traffic is reduced. This is particularly useful where management of traffic speed is difficult. A light spray of binder can also be used to extend the life of a bituminous surfacing. This is particularly useful where a surfacing is showing signs of bitumen ageing by fretting or cracking.

These applications may be referred to as Fog Sprays and Enrichment Sprays.

Fog Sprays.

A light spray of bitumen emulsion is ideal for improving early retention of chippings in a new dressing. The road surface is usually dampened before spraying or, if a low bitumen content emulsion (45 per cent) is available, this dampening can be omitted. Complete breaking of the emulsion must occur before traffic is allowed onto the dressing and it may be necessary to dust the surface with sand or crusher fines to prevent pick-up by traffic. If an emulsion is diluted with water to obtain a 45 per cent bitumen content (to ensure the bitumen will flow around the chippings), then the suitability of the water must be established by mixing small trial batches.

The spray rate for the diluted emulsion will depend upon the surface texture of the new dressing but the best results will be achieved if the residual bitumen in the fog spray is treated as part of the design spray rate for the surface treatment. The spray rate is likely to be between 0.4 and 0.8 litres/m^2. It is important to avoid over application of bitumen which can result in poor skid resistance.

Enrichment Sprays.

Surfaces which are showing obvious signs of disintegration through bitumen ageing can be enriched by applying stable grade anionic bitumen emulsion which has been diluted at a rate of 1:1 with water (Committee for State Road Authorities (1986). The rate of application will depend upon the texture of the surfacing and this must be determined by trial sprays, however, it is likely to be between 0.2 and 0.5 litres/m^2 of residual bitumen. Great care must be taken to avoid leaving a slippery surface and a light application of sand sized fines may be required in some cases.

10 FLEXIBLE PAVEMENT DESIGN CATALOGUE

10.1 Description of the Catalogue

The design of flexible pavements is based on the catalogue of pavement structures published in TRL's Overseas Road Note 31 but updated, improved and extended to higher traffic levels based on the latest research.

The Catalogue comprises seven charts for seven different basic structural types of pavement (Table 10.1) corresponding to distinct combinations of surfacing and roadbase materials. Each cell of each chart identifies the required thickness of the pavement layers and the materials for their construction based on cumulative traffic (Chapter 2) and the strength of the subgrade (Chapter 3).

It is worth repeating here that the subgrade strength for design is the lowest ten-percentile value of the strengths measured on the samples collected during the initial survey work at the 'worst strength condition' expected (or measured on a similar subgrade) during the year (see Chapter 3). Thus in a dry area the subgrade will be stronger than in a wet area. Research has shown that it will typically be at about OMC in 'average' conditions but in a very dry area it might be at 0.75*OMC and in a very wet area, for example flood prone, it might be saturated and hence at a soaked CBR value. Thus climate is taken into account by simply changing the subgrade class depending on expected CBR.

Table 10-1 Summary of the Material Requirements for the Design Charts

Chart	Surfacing	Roadbase	Chapters
A1	Bituminous surface treatments and Otta seals	T1 and T2 use GB2 or GB3. T3 use GB1,GB2 or GB3 T4 use GB1 or GB2 T5 and T6 use GB1	6 and 9
A2	Bituminous surface treatment and Otta seals	Composite (granular upper roadbase, hydraulically stabilised lower roadbase). T1 and T2 use GB2 or GB3 T3 use GB1,GB2 or GB3 T4 use GB1 or GB2 T5, T6 and T7 use GB1	6, 7 and 8
A3	Bituminous surface treatment and Otta seals	Hydraulically-stabilised roadbase CB1 and CB2	7 and 9
B	Thin, flexible asphalt concrete	T3 use GB1,GB2 or GB3 T4 use GB1 or GB2 T5 and T6 use GB1	6 and 9
C1	Asphalt concrete wearing course and binder course	Granular roadbase GB1	6 and 8
C2	Asphalt concrete wearing course and binder course	Composite (granular upper roadbase, hydraulically stabilised lower roadbase) GB1 and CB 1 and CB2	6, 7 and 8
D	Asphalt concrete wearing course and binder course	Asphalt concrete road base, RB1, RB2, RB3.	8

Notes 1 For low volume roads (T1, T2 and T3) under some conditions relaxed specifications can also be used (see the *Low Volume Roads Design Manual*)

All the charts provide alternate pavement structures for all subgrade classes (S1 through S6). They are not, however, suitable for all classes of traffic because some structures are not technically appropriate nor economically justified.

The designs themselves are based on a combination of empirical evidence and analytical (theoretical) considerations. Empirical evidence is strong at the lower traffic levels but at the higher levels there is a paucity of data and therefore theoretical considerations have also been used to develop the designs. These techniques are described in Appendix H but it is worth noting here that the accepted theory is, at best, a very imperfect tool usually resulting in relatively thick structures.

Charts A1, A2 and A3 are for pavement structures with a thin bituminous seal. At the lower traffic levels these charts overlap with the charts included in the ERA Low Volume Roads (LVR) Design Manual. In the LVR Manual pavement structures are also provided for use with non-bituminous surfacings at these lower traffic levels.

Chart A1 is for pavements with a surface treatment comprising a double surface dressing or an Otta seal on top of a granular road base. The sub-base is either a granular material or a weakly cemented hydraulically bound material. The thickness of the overall structure is controlled by the need to limit the strain in the subgrade and care must be taken to ensure that the roadbase is high quality for the higher traffic levels as indicated. Such structures can be used for pavement structures up to traffic category T6.

Chart A2 is similar to Chart A1 but this chart makes use of the benefits of a hydraulically stabilised lower roadbase. The unbound layer is generally thinner than in Chart A1 because the main strength is provided by the stabilised layer, but the total construction cost is likely to be higher due to the cost of the cement. Such structures have performed well and are generally considered more reliable than their unbound granular counterparts. This is partly because the cemented layer provides a good foundation for compacting the layers above and because it is also more tolerant of water. The subgrade strain is low with such a structure but, nevertheless, a capping layer is required to support the cemented layer and for carrying construction traffic. It should be noted that cement stabilisation may introduce significant risks on site due to delays in the supply of cement, and considerable care is required in the curing of the cemented layer.

Chart A3 shows designs using a hydraulically stabilised roadbase. It uses the same surfacings as Chart A1 and A2 but the roadbase comprises only a cement stabilised layer. Such a roadbase is strong but will contain fine cracks resulting from the curing and shrinkage of the cemented material. These cracks do not seriously affect the traffic carrying capacity of the pavement but they will eventually appear through the surface treatment and will need to be sealed with a maintenance reseal at some stage before further deterioration can occur. The cement stabilised roadbase and sub-base are efficient load spreading layers and subgrade strains will not exceed the critical values.

Chart B shows designs utilising a thin HMA surfacing. The structures themselves are similar to those in Charts A1 and A2 because thin surfacings add very little to the overall structural strength of the pavement. A flexible HMA surfacing such as hot rolled asphalt (see Chapter 8) is suitable for the lower traffic levels. However, experience in the Republic of South Africa has shown that a thin continuously graded HMA surfacing (e.g. the nominal 9.5mm maximum stone size in Table 8.7) works well provided that construction and mix design are carried out to high standards, and is now the favoured option. The importance of providing a very stiff supporting structure beneath the thin continuously graded asphalt concrete cannot be over emphasised. A strong aggregate roadbase is required and a stabilised sub-base is preferred. Chart B uses a thin asphalt concrete surfacing.

Chart C1 is a common design using a high quality structural surfacing of asphaltic concrete (wearing course and binder course) on a granular roadbase. Empirical evidence indicates that fatigue cracking of the asphaltic concrete resulting in cracks emanating from the lower surface of the AC is very rare, but such a structure should still be designed to prevent this possibility. Fatigue cracking is most sensitive to the thickness and stiffness of the asphaltic concrete itself and relatively insensitive to the thickness of the unbound layers below, hence the asphalt layer must be quite thick for high traffic levels.

Chart C2 is similar to Chart C1 but makes use of the benefits of a hydraulically stabilised lower roadbase thereby providing good support for upper layers. The upper roadbase of granular material prevents any shrinkage cracks in the cemented lower roadbase from causing sympathetic cracking in the asphaltic concrete Such structures have performed very well and are generally considered to be more reliable than their unbound granular counterparts. This is partly because the cemented layer provides a good foundation for compacting the layers above and because it is also more tolerant of water.

Chart D is another traditional design that also reflects the difficulty of preventing fatigue failure in the asphaltic concrete. In this solution, sometimes called a full-depth AC pavement, the roadbase is a bitumen-stabilised layer and the surfacing is also an asphaltic concrete. However, in tropical areas and for heavy traffic, the bituminous roadbase must be of relatively high specification (Chapter 8). The thickness of asphaltic material for the higher traffic levels is in the range that is considered to be ‘long-life’ (Nunn et al, 1997). In other words, no fatigue failure is ever likely to occur; all cracking will be ‘top-down’ and rehabilitation should consist solely of milling off the top 30 - 50mm of aged and brittle material and replacing it.

10.2 Use of the Catalogue

Although the thicknesses of layers should follow the design charts whenever possible, some limited substitution of materials between sub-base and selected fill is allowable based on the structural number principles outlined in the AASHTO Guide for Design of Pavement Structures and Appendix H. Where substitution is allowed, a note is included with the design chart.

Although a range of ‘qualities’ of granular material are specified within the charts, when they are used for high traffic levels (above T5) and even though they may be underneath high quality asphaltic concrete, it is important that they are of the highest quality crushed stone. For low traffic levels from T1 to T3 in Charts A1, A2 and A3, a gravel roadbase (GB3) may be considered.

For hydraulically (cement or lime) stabilised materials, the charts define the layers with different symbols and thereby indicate the underlying assumptions regarding the strength of material.

The choice of chart will depend on a variety of factors but, if possible, should be based on minimising total transport costs (see Section 1.6). Factors that will need to be taken into account in a full evaluation include:

i) the likely level and timing of maintenance;

ii) the probable behaviour of the structure;

iii) the experience and skill of the contractors and the availability of suitable equipment;

iv) the cost of the different materials that might be used;

v) other risk factors.

It is not possible to give detailed guidance on these issues in this manual. The charts have been developed on the basis of reasonable assumptions concerning the first three of these and therefore the initial choice should be based on the local costs of the feasible options.

If any information is available concerning the likely behaviour of some of the structures under the local conditions, then it is possible that a whole life cost analysis can also be carried out to select the most appropriate structure. However, it is unlikely that performance information will be available for many of the options until much wider design and construction experience has been gained within ERA and the PMS has been operating for a considerable number of years to record performance data. Indeed, it is likely that to appreciate the benefits of some of the structures, the Directorate of Research and Development will need to construct some full scale demonstration trials.

Therefore, for many roads, especially those that are more lightly trafficked, local experience will dictate the most appropriate structures and sophisticated analysis will not be warranted.

10.3 Design Example

An example of traffic calculations was given in Chapter 2 for a particular section of a trunk road. In the example, a total traffic for design of about 20 million ESAs was obtained and this corresponds to class T8.

The subgrade strength has been estimated to be in the range 5 to 7 percent under the worst conditions anticipated (Chapter 3). Portions of the alignment which exhibit higher strength are so limited in number and extent that it makes it impractical to consider several designs. The subgrade strength class to be assigned to this project is therefore S3 (cf. Table 3.1 and Figure 10.1).

The following preliminary information has been derived from the investigations and simple cost comparison:

- The materials which may be considered for cement- or lime-stabilisation have relatively low percentages of fines and low plasticity, thus making cement-stabilisation more promising.
- Granular sub-base materials may not be available in sufficient quantities.
- All other materials entering the composition of the possible pavement structures are available, albeit in various quantities and associated transport/construction costs.

Based on the above, and with the T8/S3 combination of traffic and subgrade strength classes, the design charts indicate the possible alternate pavement structures given in Table 10.2.

Analyses of recent contracts, production costs, hauling distances and associated costs have established relative costs for the various alternate pavement layers as shown in Table 10.3. With these elements, the relative costs of the possible alternate pavement structures are evaluated as shown in Table 10.4.

This very simplified example indicates that structure 3 is the least expensive despite the greatest thickness of asphaltic concrete in the structure. Structure 2 should have been competitive but the cost of the cement-stabilised layer is higher than expected. It is possible that there was not enough material available hence additional haulage costs were needed to bring material from a greater distance.

All of these structures ought to perform well but they are not expected to deteriorate in the same way. Local experience is required for each type to calibrate performance models and allow more accurate whole life cost principles to be used to identify the true best value options.

Table 10-2 Design Example - Possible Pavement Structures

	Design Chart No.	**C1**	**C2**	**D**
Pavement Components	Possible Structures	Structure 1	Structure 2	Structure 3
Surfacing				
Asphalt Concrete) (1)		150mm	150 mm	90 mm
Roadbase:				
Asphaltic Concrete roadbase		—	—	170 mm
Crushed Stone		200 mm	150 mm	—
Cement stabilised (2.5 MPa UCS)		—	225 mm	—
Granular sub base		325 mm	125 mm	175 mm
Selected fill		—	—	150 mm

Note: The asphaltic concrete surfacing is usually made up of 40mm of wearing course and 50 mm of binder course. An additional layer of binder or roadbase quality asphaltic concrete may be required for surfacings exceeding 90 mm.

Table 10-3 Design Example - Relative Unit Costs of Materials

Material	Thickness (mm)	Relative unit cost
Asphalt Concrete wearing course	40	0.34
Asphalt Concrete binder course	50	0.40
Asphaltic Concrete basecourse	60	0.45
Asphalt roadbase	170	1.28
Crushed stone roadbase	150	0.82
	200	1.0
Cement stabilised roadbase, 2.5 MPa	225	0.63
Granular sub base	325	0.60
	175	0.33
	125	0.25
Select fill/capping	150	0.19

Table 10-4 Relative Costs of the Alternative Pavement Structures

Alternative Pavement.	Description	Relative Unit Cost
1	40 mm wearing course	0.34
	50 mm binder course	0.40
	60 mm asphalt base course	0.45
	200 mm crushed stone roadbase	1.0
	325mm of sub-base	0.6
	TOTAL	**2.79**
2	40 mm wearing course	0.34
	50 mm binder course	0.40
	60 mm asphalt base course	0.45
	150 mm crushed stone roadbase	0.82
	225 mm of cement stabilised sub-base	0.63
	150 mm of capping	0.19
	TOTAL	**2.83**
3	40 mm wearing course	0.34
	50 mm binder course	0.40
	170 mm asphalt roadbase	1.29
	175 mm of sub-base	0.33
	150 mm of capping	0.12
	TOTAL	**2.48**

Note 1 This is a purely hypothetical example. Under no circumstances should this example be taken as representative of current prices.

KEY TO CATALOGUE

Traffic Classes (10^6 ESA)	
T1	**< 0.3**
T2	**0.3 – 0.7**
T3	**0.7 – 1.5**
T4	**1.5 – 3.0**
T5	**3.0 – 6.0**
T6	**6.0 – 10**
T7	**10 – 17**
T8	**17 - 30**
T9	**30 - 50**
T10	**50 – 80***

Subgrade Strength Classes (Lowest 10-percentile CBR per cent)	
S1	**2**
S2	**3, 4**
S3	**5 - 7**
S4	**8 - 14**
S5	**15 - 30**
S6	**> 30**

* The T10 designs are suitable for traffic of 80 mesa and are considered 'long life' pavements. They should be used for all higher traffic levels.

	Double surface treatment or Otta seal
	Thin flexible AC surfacing
	Bituminous (HMA) wearing course and binder coarse
	Bituminous roadbase
	Granular roadbase GB 1
	Granular roadbase GB 2 and 3
	Granular sub-base GS
	Granular capping layer GC
	Cement or lime-stabilised roadbase CB 1
	Cement or lime-stabilised roadbase CB 2
	Cement or lime-stabilised sub-base CS

CHART A1: SURFACE TREATMENT, UNBOUND GRANULAR ROADBASE

Subgrade class	LV3(1)(2)	LV4(1)(2)	T3/LV5(1)	T4	T5	T6
	0.1 – 0.3	0.3 – 0.7	0.7 – 1.5	1.5 – 3.0	3.0 – 6.0	6.0 – 10.0
S1	150 175 250	150 200 275	200 200 300	200 250 300	200 300 300	225 300 300
S2	150 150 175	175 150 175	200 175 200	200 225 200	200 275 200	225 275* 200
S3	150 150 125	175 150 125	200 150 175	200 175 200	200 200 200	225 200 200
S4	150 200	175 200	200 225	200 250	225 250	225 275
S5	150 125	150 150	175 175	200 175	225 175	250 175
S6	150	175	225	250	250	275

Notes

1. For these structures more options are provided in the Low Volume Roads manual.
2. The specifications for the roadbase can be relaxed as described in the Low Volume Roads manual
3. Up to 100mm of sub-base may be substituted with selected fill provided that the sub-base is not reduced to less than the roadbase thickness or 200mm whichever is the greater. The substitution ratio of sub-base to selected fill is 1 to 1.3
4. A cement or lime-stabilised sub-base (CS) may also be used but see Section 7.7.2.

CHART A2: SURFACE TREATMENT, COMPOSITE ROAD-BASE

Subgrade class	T1 < 0.3	T2 0.3 – 0.7	T3 0.7 – 1.5	T4 1.5 – 3	T5 3 – 6	T6 6 – 10	T7 10 - 17
S1	150 150 300	150 175 300	150 200 300	150 225 300	150 275 300	150 125 150 300	150 125 175 300
S2	125 150 200	150 150 200	150 175 200	150 200 200	150 250 200	150 125 125 200	150 125 175 200
S3	125 150 125	125 150 125	150 150 125	150 175 150	150 225 150	150 125 125 150	150 125 150 150
S4	125 150	125 175	150 175	150 200	150 250	150 125 125	150 125 175
S5	125 125	125 125	150 125	150 150	150 175	150 200	150 250
S6	150	150	175	200	225	125 150	150 175

CHART A3: SURFACE TREATMENT, CEMENT-BOUND ROADBASE

Subgrade class	T1 < 0.3	T2 0.3 – 0.7	T3 0.7 – 1.5	T4 1.5 – 3	T5 3 – 6	T6 6 – 10	T7 10 - 17
S1	150 150 325	150 175 325	175 175 325	200 175 325	200 225 325	200 250 325	225 250 325
S2	150 150 225	150 175 225	175 175 225	200 175 225	200 225 225	200 275 225	225 275 225
S3	150 150 150	150 150 150	175 150 150	175 175 150	200 175 175	200 225 175	225 225 175
S4	150 150	150 175	175 175	175 125 125	200 150 125	200 200 125	225 200 125
S5	150 100	150 100	175 100	175 150	200 175	200 200	225 200
S6	150	150	175	200	225	250	275

CHART B1: THIN (50mm) FLEXIBLE AC SURFACING, GRANULAR ROADBASE

Subgrade class	T3/LV5[1]	T4	T5	T6
	0.7 – 1.5	1.5 – 3.0	3.0 – 6.0	6.0 – 10.0
S1	175 200 300	175 250 300	175 300 300	200 325 300
S2	175 175 200	175 225 200	175 275 200	200 300 200
S3	175 225	175 275	175 325	200 350
S4	175 150	175 200	175 250	200 275
S5	150 100	175 125	175 150	200 175
S6	150	175	200	225

Notes

1. For these structures more options are provided in the Low Volume Roads manual.
2. Up to 100mm of sub-base may be substituted with selected fill provided that the sub-base is not reduced to less than the roadbase thickness or 200mm whichever is the greater. The substitution ratio of sub-base to selected fill is 1 to 1.3
3. A cement or lime-stabilised sub-base (CS) may also be used but see Section 7.7.2.

CHART C1: HMA SURFACE, UNBOUND GRANULAR ROADBASE

Subgrade class	T6	T7	T8
	6 – 10	10 – 17	17 - 30
S1	100 200 225 350	125 200 250 350	150 200 300 350
S2	100 200 225 200	125 200 250 200	150 200 300 200
S3	100 200 250	125 200 275	150 200 325
S4	100 200 175	125 200 200	150 200 225
S5	100 150 150	125 175 150	150 200 150
S6	100 200	125 225	150 250

CHART C2: HMA SURFACE, COMPOSITE ROAD BASE

Subgrade class	T6 6 – 10	T7 10 – 17	T8 17 - 30
S1	100 150 200 350	125 150 250 350	150 150 125 125 350
S2	100 150 200 200	125 150 250 200	150 150 125 125 200
S3	100 150 175 125	125 150 200 125	150 150 225 125
S4	100 150 175	125 150 200	150 150 225
S5	100 150 150	125 150 150	150 150 150
S6	100 100 150	125 100 150	150 100 150

CHART D1: HMA SURFACE, BITUMEN-BOUND ROADBASE

Subgrade class	T6 6 – 10	T7 10 – 17	T8 17 - 30	T9 30 - 50	T10 50 - 80
S1	90 130 200 300	90 150 225 300	90 170 250 300	90 190 250 300	90 220 250 300
S2	90 130 200 200	90 150 200 200	90 170 200 200	90 190 225 200	90 220 225 200
S3	90 130 150 150	90 150 150 150	90 170 175 150	90 190 175 150	90 220 175 150
S4	90 130 150	90 150 150	90 170 175	90 190 175	90 220 175
S5	90 130 100	90 150 100	90 170 100	90 190 100	90 220 100
S6	90 110	90 130	90 150	90 170	90 190

Note 1 The AC surfacing comprises a wearing course and 'binder' course (Section 8.5.3)

2 A cement or lime-stabilised sub-base (CS) may also be used but (Section 7.7.2)

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Appendix A: TESTING AGGREGATES FOR USE IN HMA

A.1 Shape

A.1.1 Flakiness Index

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.

A.1.2 Aggregate Angularity

Two other properties related to the shape of the aggregate are the

(i) Coarse and Fine Aggregate Angularity; and
(ii) Flat and Elongated Particles.

Angularity

A high value of angularity (i.e. more cubical) of both coarse and fine aggregate should produce high levels of internal friction and good resistance to rutting. 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 air voids in loosely compacted aggregate smaller than 2.36mm.

Flat and Elongated Particles

This characteristic is similar to the flakiness index and is 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.

A.1.3 Hardness

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 10mm and 14mm only.

A.1.4 Aggregate Crushing Value (ACV)

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.

A.1.5 10% Fines Aggregate Crushing Test (10%FACT)

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.

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.

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

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

A.1.6 Aggregate Impact Value (AIV)

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. The AIV has considerable advantages because the equipment is simple, easily portable and does not require a large crushing press.

A.1.7 Los Angeles Abrasion (LAA)

In this test (ASTM, 1996) 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.

A.2 Durability

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.

A.2.1 Aggregate Abrasion Value (AAV)

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 chippings for surface dressings. 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.

A.2.2 Polished Stone Value (PSV)

This is a predictive measure of the susceptibility of aggregate used in wearing courses and surface dressings to polishing and hence an increased 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 with a solution of corn emery and water being 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 is obtainable only from TRL. Maintaining a supply of control and calibration aggregate in developing countries may make the test difficult to sustain.

A.2.3 Water Absorption

Aggregates with high water absorption usually indicate low durability and can also cause problems during HMA design. It can be routinely measured as part of the procedure to determine the relative densities of the various size fractions of aggregate (BSI, 1975). 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 water absorption of 2 per cent or less are considered durable. A value greater than this necessitates a soundness test to be carried out for specification compliance. 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.

A.2.4 Soundness - Sodium or Magnesium Test

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 ASTM test method (ASTM, 1990) 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 are expressed as percentage material lost during the test.

Both of these tests are severe and it is known 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. The tests are relatively time consuming and are normally used where an absolute minimum of aggregate deterioration is required such as on airfields, motorways and trunk roads. However, they are particularly useful for testing aggregate obtained from rock which is thought to be susceptible to rapid weathering such as partially degraded basalt.

A.3 Cleanliness

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 on 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.

A.3.1 Decantation Test

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 coating 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.

A.3.2 Sand Equivalent Value

This test (AASHTO, 1990) 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.

A.3.3 Plasticity Index

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, 1992). 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 apply or use as a confirmatory test.

A.4 Bitumen Affinity

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.

A.4.1 Static Immersion tests

The tests are generally unreliable both in terms of repeatability and reproducibility and are usually not quoted in any aggregate specifications used for hot mix asphalt. Their usefulness is more relevant to surface dressing design. If other suitable apparatus is unavailable, the AASHTO T182 test may be useful. In this 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 whether the percentage of bitumen left on the chippings is greater or less than 95 per cent.

A.4.2 Immersion strength tests

In the immersion strength test (Whiteoak, 1990) the Marshall stability of compacted hot mix asphalt samples is measured after immersion in water maintained at 60^{o}C for 48 hours and 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 should be attained for satisfactory resistance to damage by moisture.

A similar procedure, but with variations in temperatures and immersion times, is specified in an ASTM test (ASTM, 1996). There is some doubt as to the usefulness of the tests particularly with asphalt concrete 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 more severe test (AASHTO T283) is sometimes specified in the USA.

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

A.5 Testing Standards

The recommendations herein are based on several standards including those from the United Kingdom (British Standards Institution), Australia, South Africa and the AASHTO and ASTM from the USA. Tests, even with the same name, may not give comparable results due to small 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.

Such differences are usually not critical but can become so when contractual disputes arise or when the behaviour of a material is very sensitive to the value of the particular parameter in question. For example, great care is required in the design of HMA to ensure that the volumetric composition is accurately measured. In this case the various tests required to do this, primarily density tests, are based on volumes and weights and the methods do not usually vary significantly between authorities, but care is required to use the correct method if aggregates with water absorption of greater than 1.5% are involved. Some tests are critical and so it is important that the test originally associated with the criteria is used unless the test method from other authorities is known to give the same results.

To summarise, it is important 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 BITUMEN FOR USE IN HMA

B.1 Ageing Tests and Procedures

Tests are divided into those which are used to specify the required properties of bitumen when it is delivered and others which specify the limits of acceptable changes in bitumen properties during the various stages of the HMA production process.

B.1.1 Loss on Heating Test

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.

B.1.2 Thin Film Oven Test (TFOT)

Practical conditions are simulated better by this test in that, despite being heated in a similar manner, the bitumen samples are only about 3mm thick. The amount of hardening that takes place in this test is considered to be similar to that which occurs during storage and mixing. However, the bitumen film is still too thick and it is not possible to obtain homogeneous hardening. The test, therefore, is still far from being ideal.

Precision of the TFOT - ASTM D 1754

Loss by mass

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.

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.

B.1.3 Rolling Thin Film Oven Test (RTFOT)

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 is equivalent to the degree of hardening observed during the *mixing and laying* of HMA.

B.1.4 Bitumen Durability Test

Developed by the Australian Road Research Board (ARRB), this test is an extended version of the RTFOT which has been shown to simulate the *in-service* ageing of the bitumen in thin bituminous surfacing seals (surface dressings) 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^{o}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^{o}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.

B.1.5 Consistency Tests

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

Bitumen viscosity is measured using several standard test procedures to give a profile of viscosity against temperature (Table B.1).

Table B.1 Viscosity tests to assess the quality of penetration grade bitumens

Test	Test Temperature (°C)
Penetration test	Standard temperature is 25°C Lower temperature, e.g. 15°C Higher temperature, e.g. 35°C
Ring and Ball test	Determines temperature at which a standard amount of deformation occurs.
Brookfield	Typically 3 temperatures between about 100 and 160°C

The penetration test is not strictly a viscosity test; however there is a very close relationship between penetration at 25°C and viscosity. The Shell Bitumen Data Chart allows Penetration Values, Ring and Ball Softening Point and Brookfield Viscosity test results to be plotted show the relationship between viscosity and temperature and thereby to identify the most appropriate temperatures for mixing and for compaction.

B.1.6 Penetration Test

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^{o}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;

$$Log\,(pen) = AT + C$$

Where A is the temperature susceptibility, and

C is a constant

The value of A varies from 0.015 to 0.06, showing there may be a considerable difference between the temperature susceptibility of different bitumens. To quantify this in a more convenient way, the Penetration Index (PI) is defined as follows:

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

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:

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

Precision of the Penetration Test at 25^{o}C - ASTM D 5

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.

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.

Precision of the Percentage of Retained Penetration Test

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%.

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%.

B.1.7 Softening Point Test

A number of specifications for penetration grade bitumens also require the softening point of the binder. 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^{o} C higher than those recorded with the other methods.

The consistency of bitumen at the softening point temperature has been measured in terms of penetration and been found to be 800. Therefore replacing log (pen T_2) by log($T_{R\&B}$) in the equation above and (pen at T_2) by 800 the following equation is obtained that can be used as an alternative, though slightly less accurate, method of deriving the PI of a bitumen.

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

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

The value of Penetration of 800 at the $T_{R\&B}$ 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 $T_{R\&B}$. Also the relationship may not be true for aged bitumens or for modified bitumens. The most reliable PI results are likely to be obtained when the Penetration test can be carried out at two temperatures with good reproducibility.

A penetration ageing index, which is the ratio of the penetration at 25°C after ageing to that of the original bitumen, can also be used for comparison purposes.

Precision of the Softening Point - ASTM D 36

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^{o}C.

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^{o}C.

B.1.8 Fraass Breaking Point Test

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^{o}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, at fracture, the bitumen has a stiffness of $2.1x10^9$ Pa.

B.1.9 Measurement of Bitumen Viscosity

Because 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^{o}C and a minimum value at 135^{o}C. A temperature of 60 ^{o}C was chosen because it is approximately the maximum temperature of in-service asphalt surfacings and 135^{o}C because it is approximately the temperature at mixing and lay down.

Two types of viscosity test at 60^{o}C are in common use 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 standard calibrating oil. They work in a similar way by measuring the time taken for the binder, at 60^{o}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 poise.

Penetration grade bitumens are sufficiently fluid at 135^{o}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 is the *Zeitfuchs Cross-Arm Viscometer*, which again is calibrated with standard calibrating oils. The viscometer is housed in an oil bath maintained at 135^{o}C and the time taken for the bitumen to flow between two points, under gravity, is recorded. The time, again 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^{o}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.

Viscometers other than capillary viscometers are also in common use. One such rotary instrument is the *Brookfield viscometer*. 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.

A fundamental method of measuring viscosity is *the sliding plate viscometer*. This apparatus applies the definition of absolute (or dynamic) viscosity, i.e. it takes the shear stress (Pa) applied 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.

B.1.10 Ductility

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^{o}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 (cm) at which the thread breaks is designated the ductility of the bitumen.

B.1.11 The Bitumen Test Data Chart

The bitumen test data chart (BTDC) is a very useful method of showing the viscosity characteristics of a bitumen on one chart and defining the best temperatures for mixing, for laying and for compaction. 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 B.1 shows the BTDC with typical temperature-viscosity relationships for three penetration grade bitumens.

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 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.

There are optimum values of bitumen viscosity for the mixing and compaction of dense bituminous mixes. These are illustrated in Figure B.2 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 the optimum viscosity is between 2 and - 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.

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 B.3. The three classes are Class S (for straight line) bitumens which comprise penetration grade bitumens with low 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.

B.1.12 Superpave tests

The consistency tests described above are those most commonly used and generally require relatively low cost equipment. In comparison, the Superpave design procedure (Asphalt Institute, 1997) now calls for a more extensive range of viscometer tests 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 B.2 lists the test procedures, the purpose of the tests and any pre-conditioning of the binder used in the tests.

Table B.2 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

B.1.13 Purity Tests

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.

Precision of the Solubility test - ASTM D 2042

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%.

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%.

B.1.14 Safety Tests

Normally bitumen is free from water as 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 $100^{o}C$. Bitumen foaming is a safety hazard and a number of specifications require the binder to be free of water and not to foam at $175^{0}C$.

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.

Precision of the Flash Point by Cleveland Open Cup - ASTM D 92

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^{o}C.

Reproducibility

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

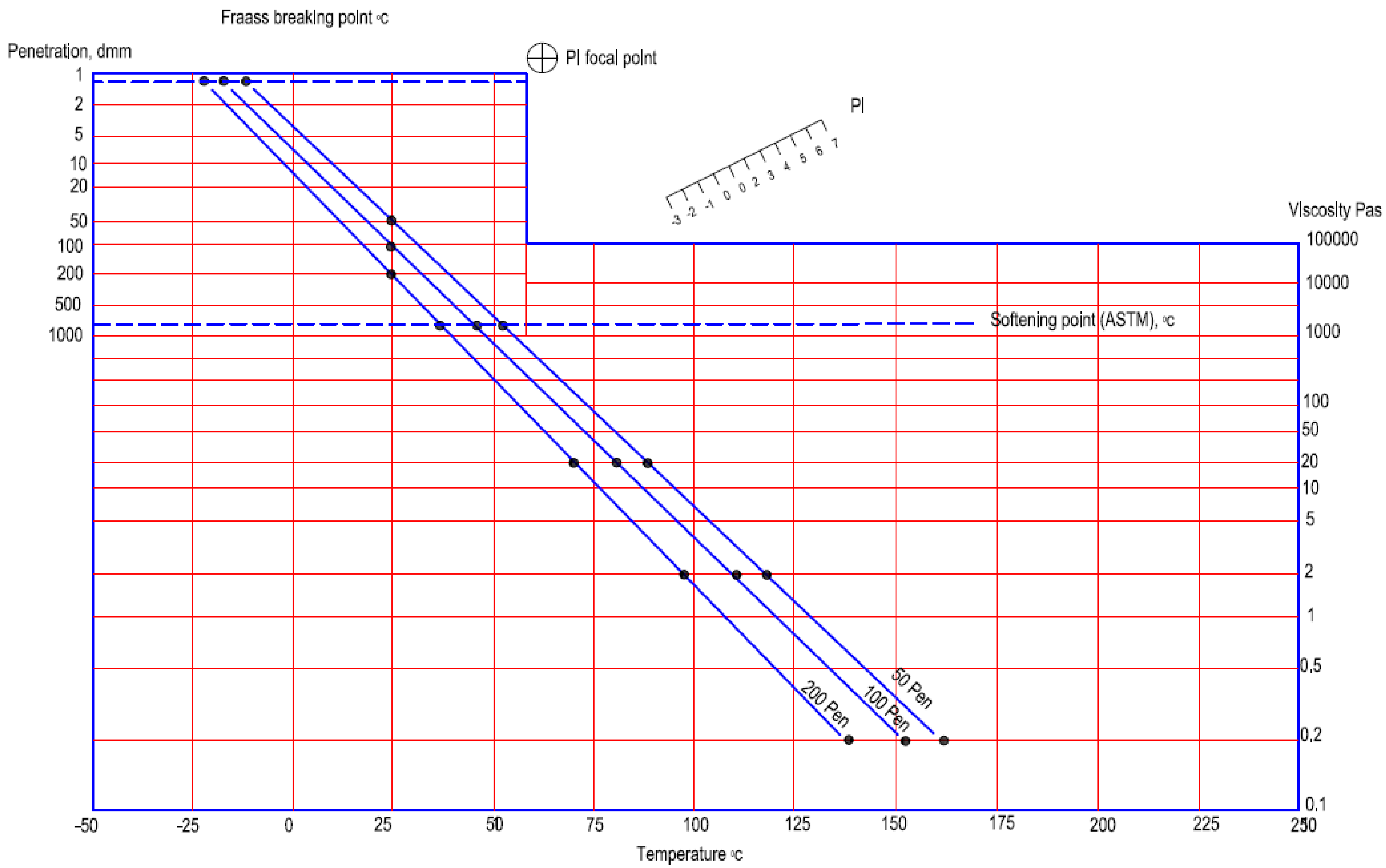


Figure B.1 A bitumen test data chart for three penetration grade bitumens (Whiteoak, 1990)

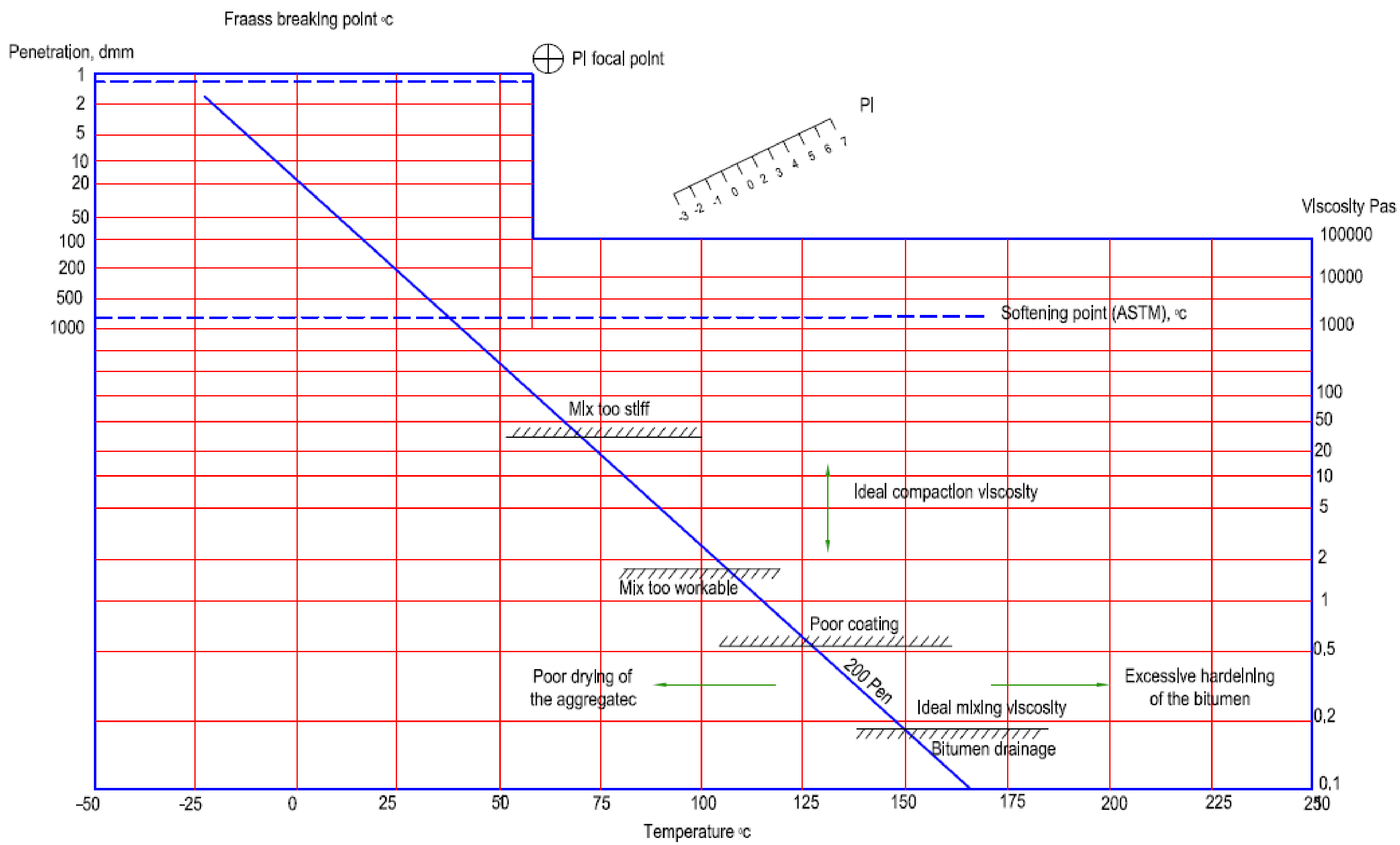


Figure B.2 A bitumen test data chart showing viscosity and therefore temperature ranges for mixing and compactions (Whiteoak, 1990)

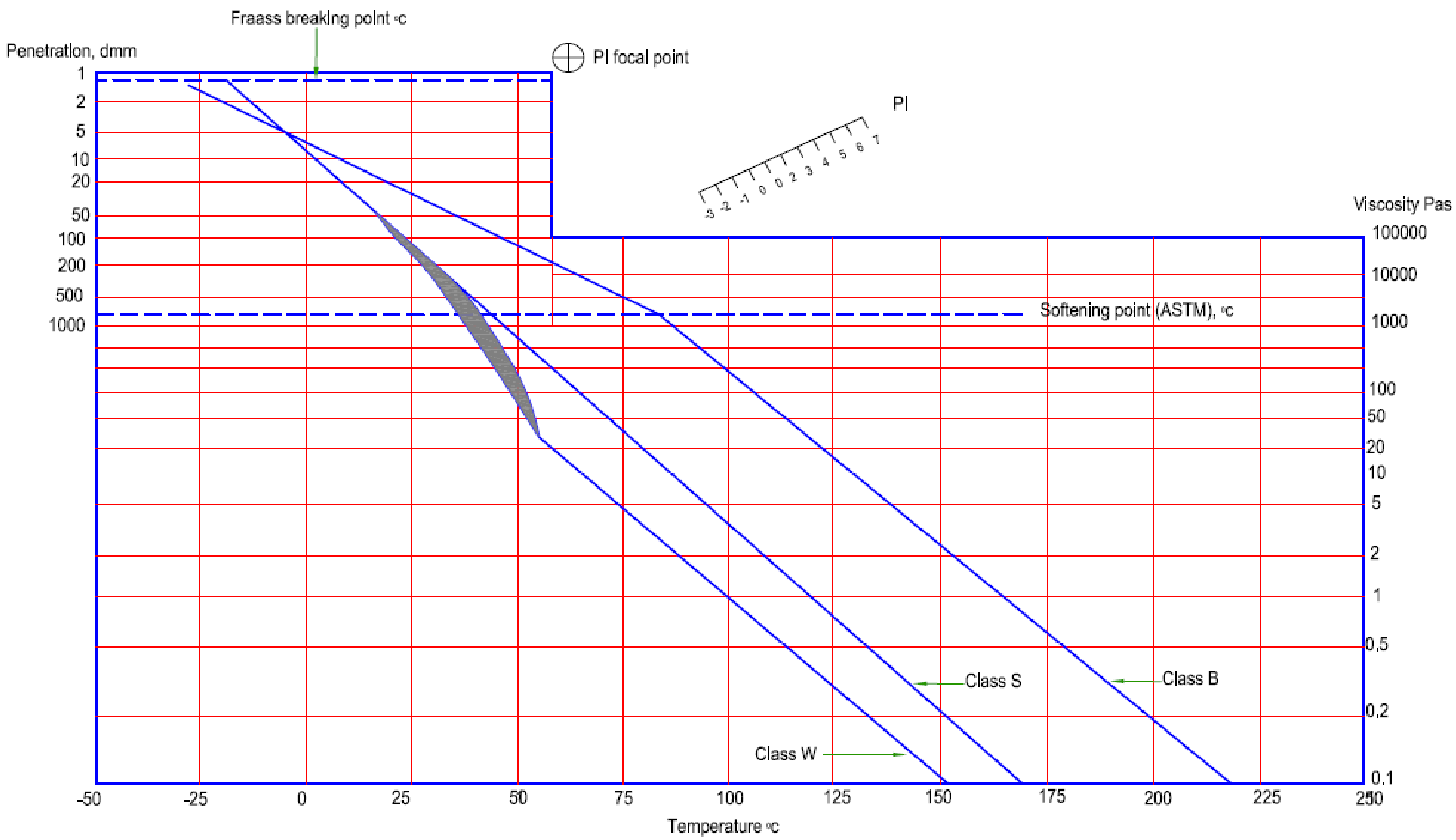


Figure B.3 A bitumen test data chart showing class S, B and W bitumens (Whiteoak, 1990)

Appendix C: MARSHALL DESIGN METHOD FOR HMA

C.1 Introduction

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.

C.2 Materials

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 allow for some wastage. The materials used must be representative of those to be used on the project.

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.

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. If samples have to be reheated, the degree of heating must be kept to a minimum and the possible effects of this reheating process should be investigated. This can be done by splitting a hot sample and carrying out Marshall tests on the hot material and also on the second sample after cooling and re-heating.

C.2.1 Aggregates

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.

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.

C.2.2 Design of aggregate grading

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.

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.

C.2.3 Bitumen

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.

C.2.4 Determination of mixing and compaction temperatures

The following properties of the bitumen are measured:-

i) Penetration at 25°C

ii) Softening point

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) (Appendix B) from which the ranges of ideal mixing and compaction temperature can be obtained.

The specific gravity of bitumen is required for the volumetric design of the mix.

C.3 Preparation of Test Samples

C.3.1 Mass of aggregate required

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 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:-

$$Adjusted\ weight = \frac{63.5 * (weight\ of\ aggregate\ used)}{specimen\ height\ (mm)\ obtained} \qquad \text{Equation C.1}$$

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 above.

C.3.2 Design bitumen content

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):

$$DBC = 0.035a + 0.04b + Kc + F \qquad \text{Equation C.2}$$

where:

DBC	=	approximate design bitumen content, per cent by 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.

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.

C.3.3 Mixing

Before mixing, the half-litre containers of bitumen are heated in an oven to the ideal mixing temperature as determined 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.

C.3.4 Compaction

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.

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.

The temperature of the mix prior to compaction must be within the determined limits (see 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 is 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.

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

C.4 Testing of Specimens

C.4.1 Bulk specific gravity determination

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

C.4.2 Stability and flow testing

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 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.

C.4.3 Determination of VIM

The maximum specific gravity of the mixes at each bitumen content must be determined to enable VIM to be calculated (see below). 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.

C.4.4 Test data

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) Bulk Specific Gravity of mix v bitumen content

C.4.5 Confirmation of design bitumen content

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.

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.

C.5 Volumetric analysis

C.5.1 Determination of specific gravity for volumetric analysis.

Because it is the ***volume*** of the individual components that is important for satisfactory mix design, 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 C.1.

Table C.1 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 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	-	-

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 BSG's of which are very different and must be tested separately.

Determination of the BSG's of the aggregates is based on the oven dried weight. Accuracy of measurement is important and it is recommended that they are determined to four significant figures, i.e. three decimal places. If the BSG's 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.

The BSG's 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;

$$Gsb = \frac{P_1 + P_2 + \dots P_n}{\frac{P_1}{G_1} + \frac{P_2}{G_2} + \dots \frac{P_n}{G_n}} \qquad \text{Equation C.3}$$

where: G_{sb} = bulk specific gravity for the total aggregate
$P_1, P_2 \dots P_n$ = individual percentages by weight of aggregates
$G_1, G_2 \dots G_n$ = individual bulk specific gravities of aggregates

During the production of HMA it is essential that the plant produces the same aggregate blend as obtained in the laboratory design. Adjustments must be made if the laboratory design is expressed in terms of volume since the plant will be set up to proportion by mass.

To complete the volumetric analysis of a bituminous mix it is necessary to determine the maximum specific gravity (G_{mm}) of the loose HMA, the BSG of the compacted material (G_{mb}) and the SG of the bitumen (G_b) used in the mix.

C.6 Calculation of volumetric properties of individual components

C.6.1 Effective specific gravity of aggregate

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:

$$G_{se} = \frac{100 - P_b}{\frac{100}{G_{mm}} - \frac{P_b}{G_b}} \qquad \text{Equation C.4}$$

where: G_{se} = effective specific gravity of aggregate;
G_{mm} = maximum specific gravity of mixed material (no air voids);
P_b = bitumen content at which ASTM D2041 test (G_{mm}) was performed, percent by total weight of mixture;
G_b = specific gravity of bitumen

C.6.2 Maximum specific gravity of mixtures with different bitumen contents

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.

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.

The 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:

$$Gmm = \frac{100}{\frac{Ps}{G_{se}} + \frac{Pb}{G_b}} \qquad \text{Equation C.5}$$

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

C.6.3 Bitumen absorption

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

$$P_{ba} = \frac{100(G_{se} - G_{sb})G_b}{G_{se}G_{sb}} \qquad \text{Equation C.6}$$

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

C.6.4 Effective bitumen content of the mix

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

$$P_{be} = P_b - \frac{P_{ba}P_s}{100} \qquad \text{Equation C.7}$$

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

C.6.5 Percent voids in mineral aggregate (VMA)

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:

$$VMA = 100 - \frac{G_{mb}P_s}{G_{sb}} \qquad \text{Equation C.8}$$

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

C.6.6 Percent air voids in a compacted mix

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

$$VIM = 100\left(\frac{G_{mm} - G_{mb}}{G_{mm}}\right) \qquad \text{Equation C.9}$$

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

C.6.7 Percent voids filled with bitumen (VFB) in a compacted mix

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

$$VFB = 100\left(\frac{VMA - VIM}{VMA}\right) \qquad \text{Equation C.10}$$

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

C.7 Worked Example for Calculating the Volumetric Components of HMA

C.7.1 Example properties of materials and HMA

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 C.2.

Table C.2 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 (e.g.. cement)	2 (P_6)	3.120 (G_6)
TOTAL	100	

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

The Marshall data obtained from samples using the aggregate proportions shown in Table C.2 at five bitumen contents are detailed in Table C.3.

The G_{mm} (ASTM D2041) of the loose material containing 4.5% and 5.0% of bitumen, i.e. the two bitumen contents nearest to the optimum, was determined as 2.531 and 2.511 respectively.

Table C.3 Marshall properties

Per cent bitumen by 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	9
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	8
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**

C.8 Calculation of volumetric composition

C.8.1 Bulk Specific Gravity of total aggregate (G_{sb})

Substituting the data in Table C.2 into Equation C.3;

$$G_{sb} = \frac{5+10+25+48+10+2}{\frac{5}{2.727}+\frac{10}{2.731}+\frac{25}{2.732}+\frac{48}{2.691}+\frac{10}{2.584}+\frac{2}{3.120}} = \frac{100}{36.996} = 2.703$$

C.8.2 Effective Specific Gravity of total aggregate (G_{se})

G_{se} is calculated by substituting the values of G_{mm} at the two test bitumen contents of 4.5% and 5.0% and the specific gravity of bitumen in Equation C.4;

With 4.5% bitumen content;

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

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**

C.8.3 Maximum Specific Gravity of mixes with different bitumen contents

By using Equation C.4 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.

With 3.5% bitumen content: $$G_{mm} = \frac{100}{\frac{96.5}{2.717} + \frac{3.5}{1.03}} = \frac{100}{38.915} = 2.570$$

With 4.0% bitumen content: $$G_{mm} = \frac{100}{\frac{96.0}{2.717} + \frac{4.0}{1.03}} = \frac{100}{39.216} = 2.550$$

With 5.5% bitumen content: $$G_{mm} = \frac{100}{\frac{94.5}{2.717} + \frac{5.5}{1.03}} = \frac{100}{40.121} = 2.493$$

C.8.4 Bitumen absorption (P_{ba})

Using Equation C.6 the bitumen absorption value can be determined:

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

C.8.5 Effective bitumen content (P_{be})

Using Equation C.7 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.

C.8.6 Voids in Mineral Aggregate (VMA)

Using Equation C.8 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 4.0, 4.5, 5.0 and 5.5% bitumen content.

C.8.7 Air voids in compacted mix (VIM)

Using Equation C.9 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 4.0, 4.5, 5.0 and 5.5% bitumen content.

C.8.8 Voids filled with bitumen (VFB)

Using Equation C.10 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 4.0, 4.5, 5.0 and 5.5% bitumen content.

C.8.9 Presentation of Data

The complete volumetric and Marshall data is summarised in Table C.4.

Table C.4 Summary of volumetric and Marshall data

Bitumen content (%)	Bulk SG of specimen (G_{mb})	Max SG of loose mix (G_{mm})	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

The test properties in Table C.4 are presented graphically in Figure C.1.

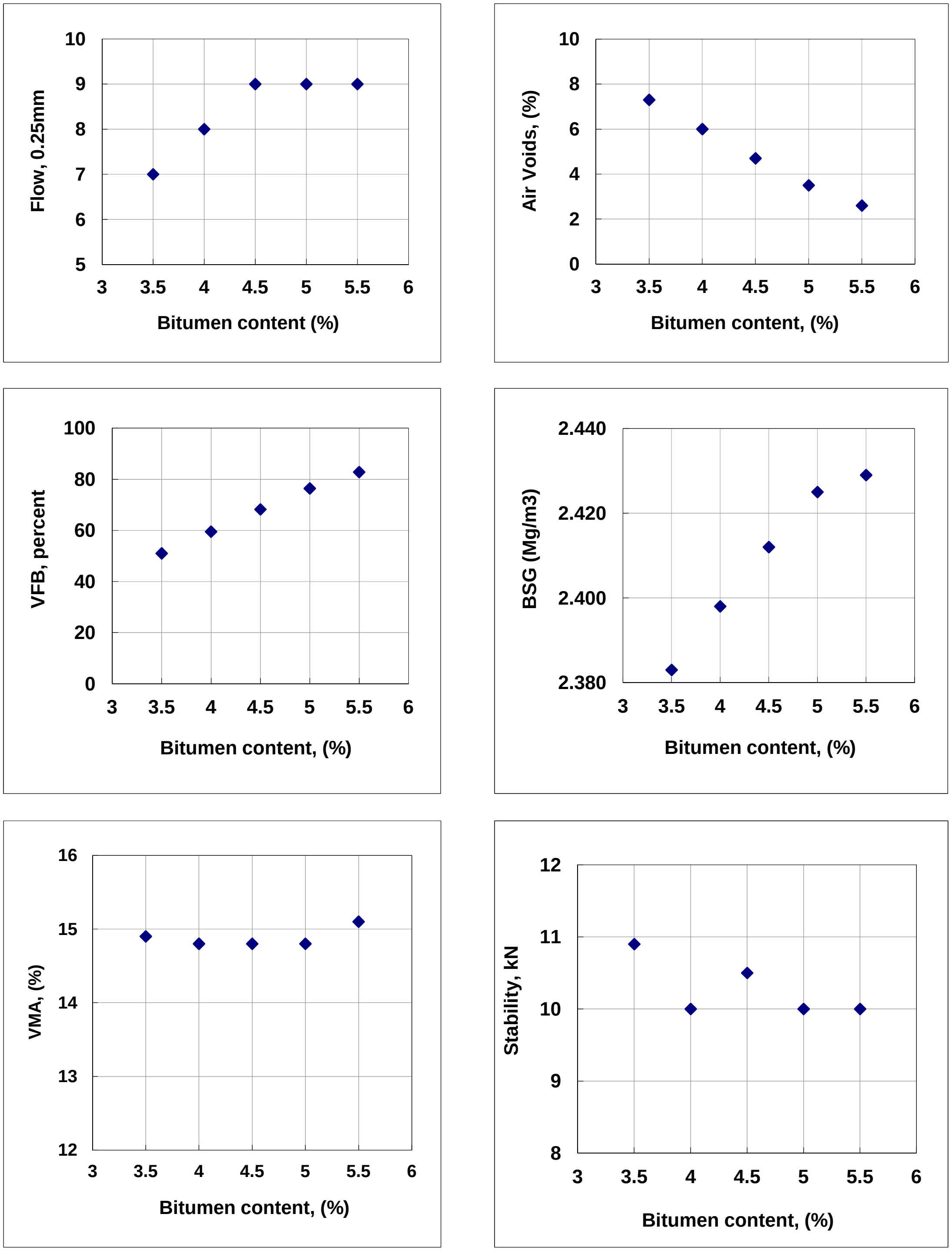


Figure C.1 Graphical representation of mix test properties

C.9 Trends and relationships of test data

By examining the test properties graphically (see Figure C.1) 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 BSG of the total mix increases with increasing bitumen content but it 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 increasing 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.

C.9.1 Determination of design bitumen content

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. All of the calculated and measured mix properties at this bitumen content are determined by interpolation from the graphs shown in Figure C.1. The individual properties are then compared to the mix design criteria as specified in MS-2 (Asphalt Institute, 1994).

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 C.5.

Table C.5 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

C.9.2 Selection of final mix design

The final selected mix design is usually the most economical one that will satisfy all of the established criteria stated in MS-2. 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 C.6 shows the mix properties, the design criteria (as specified in MS-2) and the range of bitumen content over which compliance with the criteria is achieved (obtained from the graphs in Figure C.1). This data can be presented in a bar chart such as that shown in Figure C.2 below, which clearly illustrates the effect of variations in bitumen content on the design parameters.

Table C.2 Per cent bitumen range complying with MS-2 mix property criteria

Mix property	MS-2 criteria	% range of bitumen content giving compliance with MS-2 criteria
VIM	3 - 5 %	4.4 – 5.3
VMA	13% minimum	3.5 – 4.9 (remaining on ‘dry’ side)
VFB	65 – 75 %	4.3 – 4.9
Stability	8 kN minimum	3.5 – 5.5

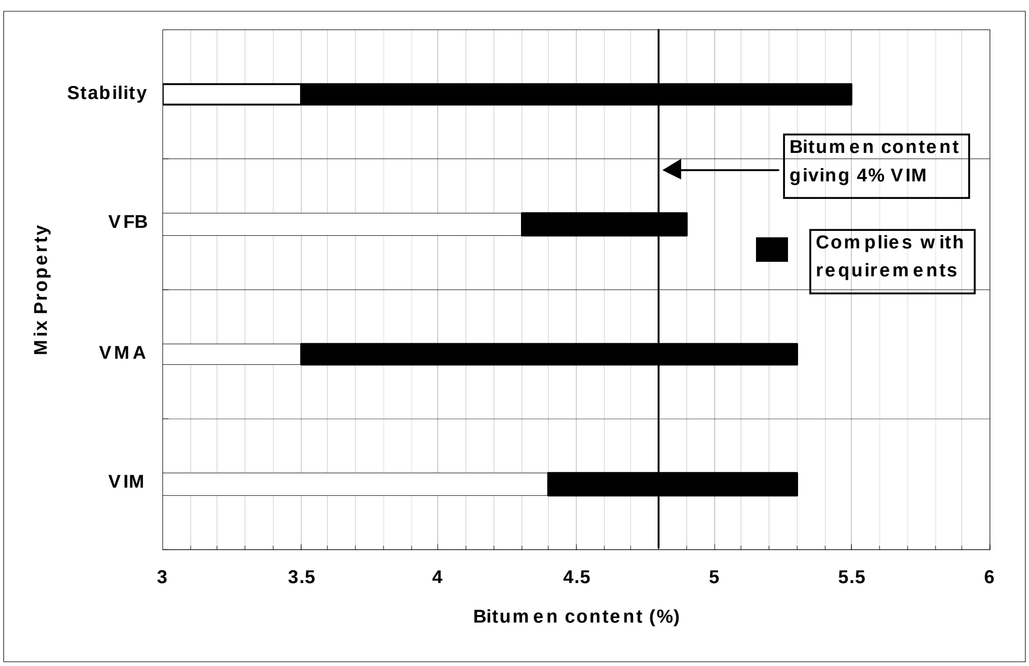


Figure C.2 Acceptable bitumen range complying with design criteria

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. If the design traffic is near to the lower end of the appropriate category, in this case $1x10^6$ ESA, then it may be acceptable to specify a target bitumen content of 4.8 per cent. However, if the design traffic is nearer to the upper limit of $5x10^6$ ESA, then a lower target bitumen content of 4.6 per cent may be more appropriate but in this case the maximum VIM will probably be exceeded in a small percentage of the surfacing after trafficking.

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 C.1). In this case it would be wise to carry out additional tests to confirm the mix properties.

After completing the additional tests, the possibilities are;

(i) that the addition test results confirm that the bitumen content giving 4 per cent VIM is acceptable;

(ii) that a design bitumen content of 4.6 per cent would minimise the risk of plastic deformation; and

(iii) that the aggregate particle size distribution should be adjusted further away from the maximum density line to give slightly more VMA.

On any project pre-construction compaction trials are an essential part of the mix design process and should be used to ensure that the mix is satisfactory.

C.9.3 Confirmation of Volumetric Analysis

It is important to ensure that the volumetric analysis is correct. 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 9 microns can be used as a guide when assessing the suitability of a particular design bitumen content. However, this must be considered together will all evidence from the laboratory design testing and the pre-construction compaction trials before confirming the properties of the target mix.

C.9.4 Bitumen film thickness

Bitumen film thickness can be estimated using the following formula (TRH 8:1987);

$$F = \frac{B_e}{100 - B} * \frac{1}{A} * \frac{1}{S} * 10^6 \qquad \text{Equation C.11}$$

where:

	F	=	Film thickness
	B_e	=	Effective bitumen content of HMA (% by mass of mix)
	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 (m^2/kg)

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

$$(2 + 0.02a + 0.04b + 0.08c + 0.14d + 0.3e + 0.6f + 1.6g) * 0.20482$$

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

C.9.5 Effect of Compaction on Design Bitumen Content

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 and, as current traffic loadings increasingly exceed $1x10^6$ ESA, the lower limit in MS-2 (Asphalt Institute, 1994) which defines heavy traffic.

Figure C.3 indicates the effect which an underestimation of secondary compaction can have on the final properties of a dense wearing course.

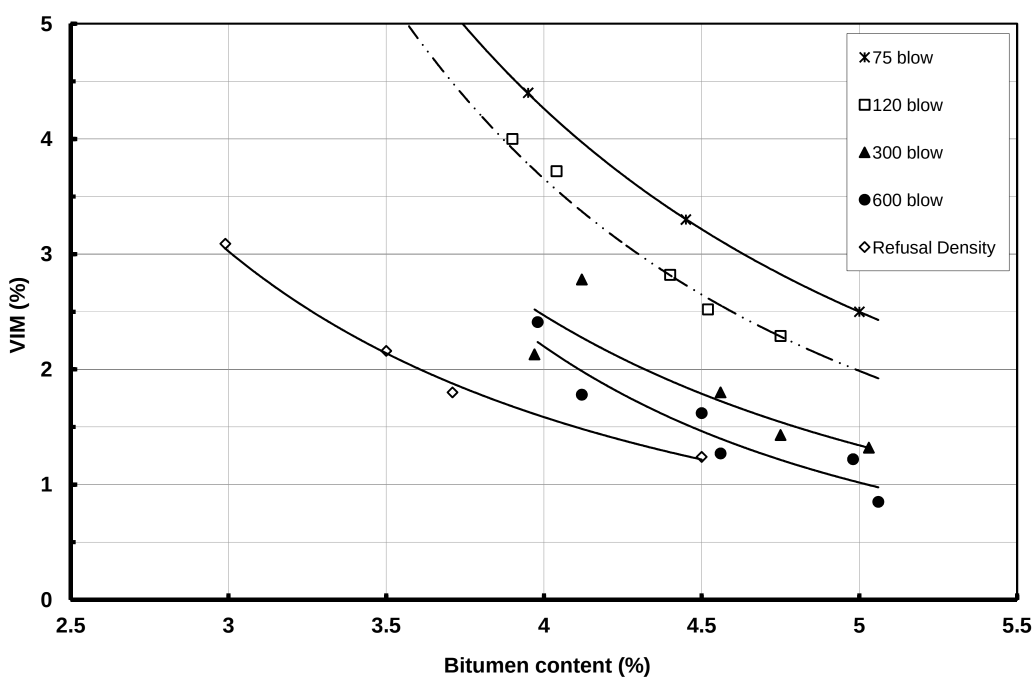


Fig C.3 Effect of compaction on mix properties

The results demonstrate how plastic deformation can occur when secondary compaction under traffic is underestimated resulting in a reduction in VIM to less than the critical value of 3 per cent. For instance, using 75 blows 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 a bitumen content of 4.15 percent, VIM will be reduced to approximately 3.3 per cent and 2.2 per cent respectively.

It is recommended that for design traffic in excess of $5x10^6$ ESA the design VIM should be 5 per cent at 75 blows compaction. Applying these criteria to the data in Figure C.3 indicates that the design bitumen content would 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.

It is noticeable 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 also that field compaction trials showed that the mix is sufficiently workable.

Appendix D: AN INTRODUCTION TO SUPERPAVE

D.1 Background

Superpave mix design is described in the following manuals produced by the Asphalt Institute (AI, 1996):

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

(ii) Superpave Mix Design. Superpave Series No. 2 (SP-2).

Only an outline of the procedure is given here to give an indication of the general methodology.

D.2 Materials for Superpave

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 outlined in paragraphs D.2.3 and Tables D.1 and D.2.

Mix acceptance is based on volumetric compositional and compaction characteristics which are specified for different levels of 20-year traffic loading expressed in terms of equivalent standard axles.

D.2.1 Selection of grade of bitumen

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.

For example, the bitumen grade required for a high design temperature of 52°C and a low design temperature of -10°C is designated 52 -10. Further adjustments are recommended to take account of severity of traffic loading conditions since the basic binder selection is based on the assumption of typical fast free-flowing traffic. Final selection of the grade of bitumen then takes into account the actual designations of standard grades of binders as given in AASHTO specification MP1.

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.

D.2.2 Performance tests for bitumen

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.

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.

D.2.3 Aggregate properties

The properties specified, 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.

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 D.1 and D.2.

D.3 Compaction for Superpave mix design

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.

Table D.1 Particle size distribution for Superpave HMA wearing courses

Sieve size (mm)	**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**	
	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
0.6			16.7	20.7			19.1	23.1			23.5	27.5
0.3			13.7	13.7			15.5	15.5			18.7	18.7
0.075	2	8			2	10			2	10		

Table D.2 Particle size distribution for Superpave HMA roadbase and binder courses

Sieve size (mm)	**Per cent passing sieve size Nominal maximum 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
0.6			11.7	15.7			13.6	17.6
0.3			10	10			11.4	11.4
0.075	0	6			1	7		

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

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).

Table D.3 Superpave gyratory compaction effort

Design traffic (esa x 10^6)	Compaction parameters		
	N initial	N design	N maximum
< 0.3	6	50	75
0.3 – 3 (Note 2)	7	75	115
3 – 30	8	100	160
>30	9	125	205

Notes

1 Design traffic is the anticipated 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 D.4 Superpave HMA design requirements

<table>
<tr><th rowspan="3">Design traffic (esa x10^6)[1]</th><th colspan="3" rowspan="2">Required relative density (Per cent of theoretical maximum specific gravity)</th><th colspan="5">Minimum Voids in Mineral Aggregate (VMA) (per cent)</th><th rowspan="3">Range of Voids Filled with Bitumen (%)</th><th rowspan="3">Range of Filler Binder ratio</th></tr>
<tr><th colspan="5">Nominal maximum aggregate size (mm)</th></tr>
<tr><th>N initial</th><th>N design</th><th>N max</th><th>37.5[2]</th><th>25[3]</th><th>19</th><th>12.5</th><th>9.5[4]</th></tr>
<tr><td>< 0.3</td><td>≥ 91.5</td><td rowspan="5">96.0</td><td rowspan="5">≥ 98.0</td><td rowspan="5">11.0</td><td rowspan="5">12.0</td><td rowspan="5">13.0</td><td rowspan="5">14.0</td><td rowspan="5">15.0</td><td>70[3]-80</td><td rowspan="5">0.6-1.2[5]</td></tr>
<tr><td>0.3 - 3</td><td>≥ 90.5</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 > 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.

D.4 Preparation of mix design samples

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

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.

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.

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.

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

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.

After compaction each sample is allowed to cool partially 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.

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

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

D.5 Moisture sensitivity

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.

D.6 Construction of power grading chart

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.

Typical sieve sizes referred to in various international and country standards are shown in Table D.5 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.

An example of a 0.45 power chart for a particle size distribution having a maximum aggregate size of 50mm is shown in Figure D.1.

D.7 Comments on the Superpave Method

The method gives an impression of accuracy that must be treated with caution. Top down cracking is not specifically referred to and thermal cracking is assumed to be related to low temperature cracking which will not normally apply in tropical areas. However, thermal cracking may apply in sub-tropical or desert areas which experience cold winters or extreme diurnal temperature ranges.

The full procedures are unlikely to be in general use for many years. A complete design for one mix will take fully trained technicians approximately one month to complete and demands a very high level of competency and quality control at every stage of the process, therefore, alternative design procedures have been suggested in this manual.

Nevertheless there are several aspects of the method that can be adopted for mix design. It is, for example, generally accepted that the gyratory compactor simulates field compaction much better than drop-hammer compaction and can be adopted when using other methods of mix design.

The particle size distribution and the ‘restricted zone’ principles can also be applied. Although some mixes whose particle size distribution passes through the restricted zone are known to be successful, using the principles when dealing with a new source of aggregate can save time.

Finally, the method of specifying bitumens may become widespread and should help agencies obtain more reliable bitumens.

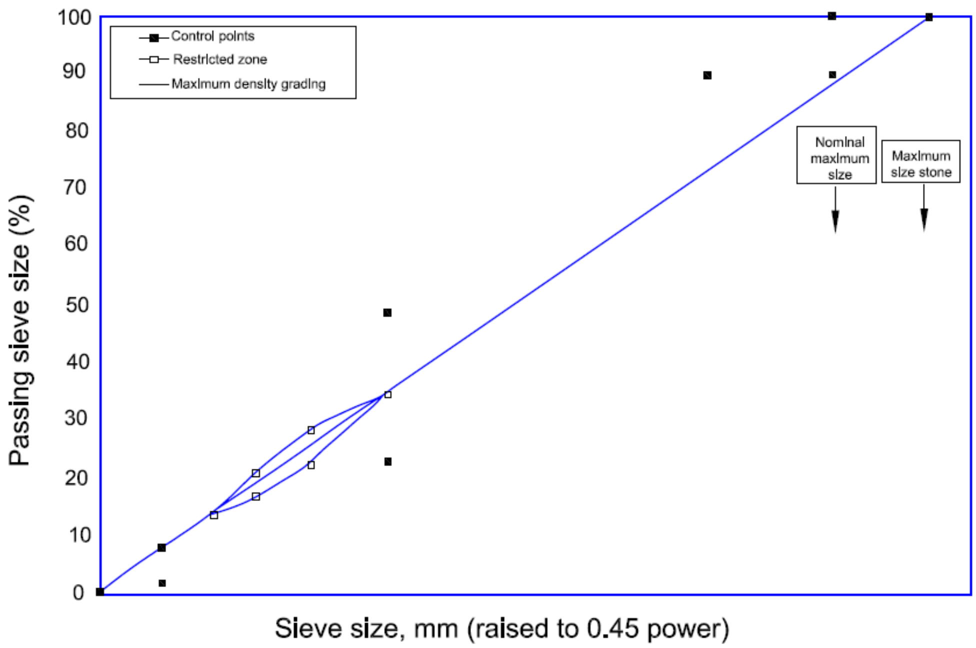


Figure D.1 Particle size distribution limits for a 37.5 mm nominal maximum size aggregate

Appendix E: Refusal Density Test and HMA Design

E.1 Equipment

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;

(i) A tamping foot with a diameter of 102mm
(ii) A tamping foot with a diameter of 146mm
(iii) 2 x shanks for the tamping feet
(iv) 8 x 152-153mm diameter split moulds
(v) 9 x base plates
(vi) A 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.

E.2 Vibrating hammer compaction

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.

E.2.1 Compaction of loose mix material

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 6 the selected maximum stone size in the mix may be influenced by the thickness of the layer to be constructed.

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.

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.

To ensure refusal density is achieved the compaction process should then immediately be repeated 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.

E.2.2 Compaction of cores to refusal density

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% of the mass of the core over a 2 hour period.

The core is then allowed to cool to ambient temperature and weighed before determining its bulk density. When the core is permeable, this 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.

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 and subjected to refusal compaction as described above. The sample is allowed to cool before removing it from the mould and then, after reaching ambient temperature, its bulk density is determined.

The Percentage Refusal Density (PRD) is calculated using the following formula;

$$PRD = \left(\frac{Bulk\ density\ of\ core}{Bulk\ density\ after\ PRD\ compaction} \right) x100 \qquad \text{Equation E.1}$$

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 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.

E.3 Refusal density design

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 described in Chapter 8 of this manual.

Sometimes there is a choice of aggregate sources or sizes that are available 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 E.1 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%, mixes with VMA less than 13% will have a very low bitumen content and will be difficult to compact.

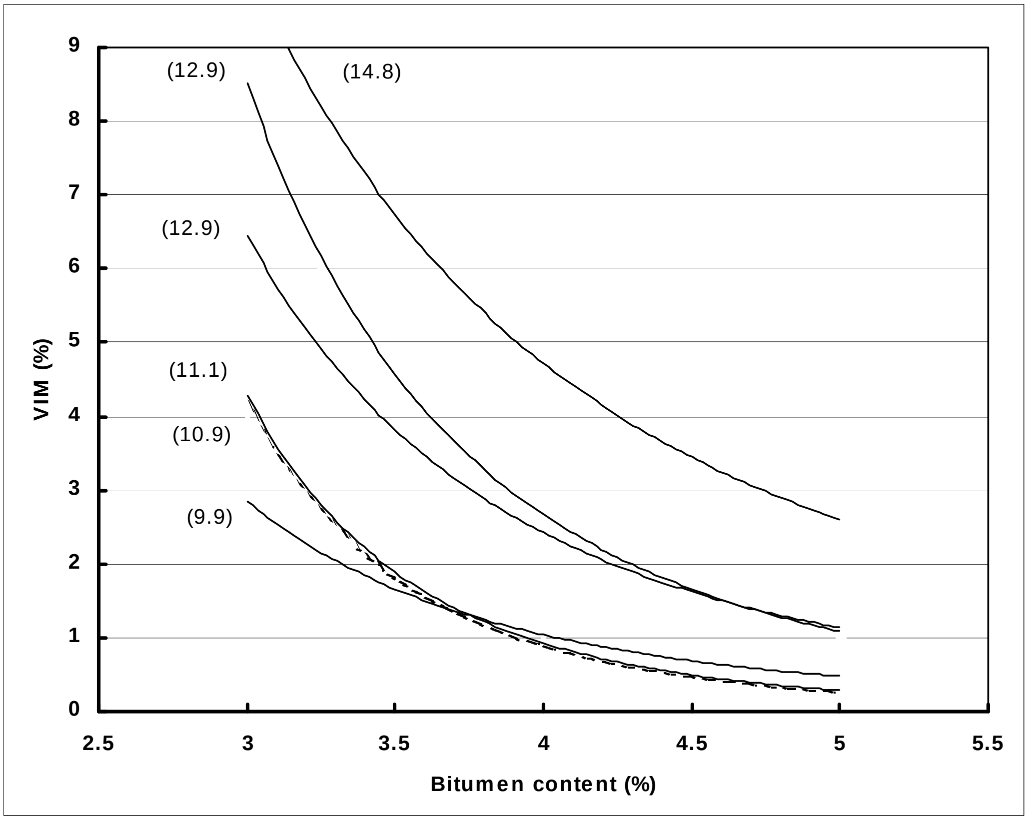


Figure E.1 Examples of VIM and VMA relationships for mixes compacted to refusal

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 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 D.2 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 tougher than the coarser mixes.

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, allowed to cool overnight then tested to determine its bulk density. The maximum specific gravity of the mixes (ASTM D2041) must also be determined (see Appendix C) so that VIM in each compacted sample can be accurately determined.

The best balance in mix properties will be obtained with the densest mix which can accommodate sufficient bitumen to make the mix workable but which is also as insensitive as possible to variations in proportioning during manufacture and 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 new mix, but omitting any material larger than 25mm.

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 absolutely essential and this level of detail will need to be developed based on local experience.

E.3.1 Compaction specifications for HMA designed to refusal density

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 its refusal density.

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.

E.3.2 Durability of HMA surfacings designed to refusal density

As described above, 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 8 to 10 per cent. It will therefore 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.

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.

Figure E.2 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 surfaced with a Cape seal as part of the construction process. The figure shows that even when sealed, dense mixtures with high bitumen content and low

VIM can age harden to penetrations of less than 30 within four years. However, this is a relatively slow rate of hardening compared with the rate that is observed in unsealed surfacing mixes that would then become very brittle and suffer from ‘top down’ cracking.

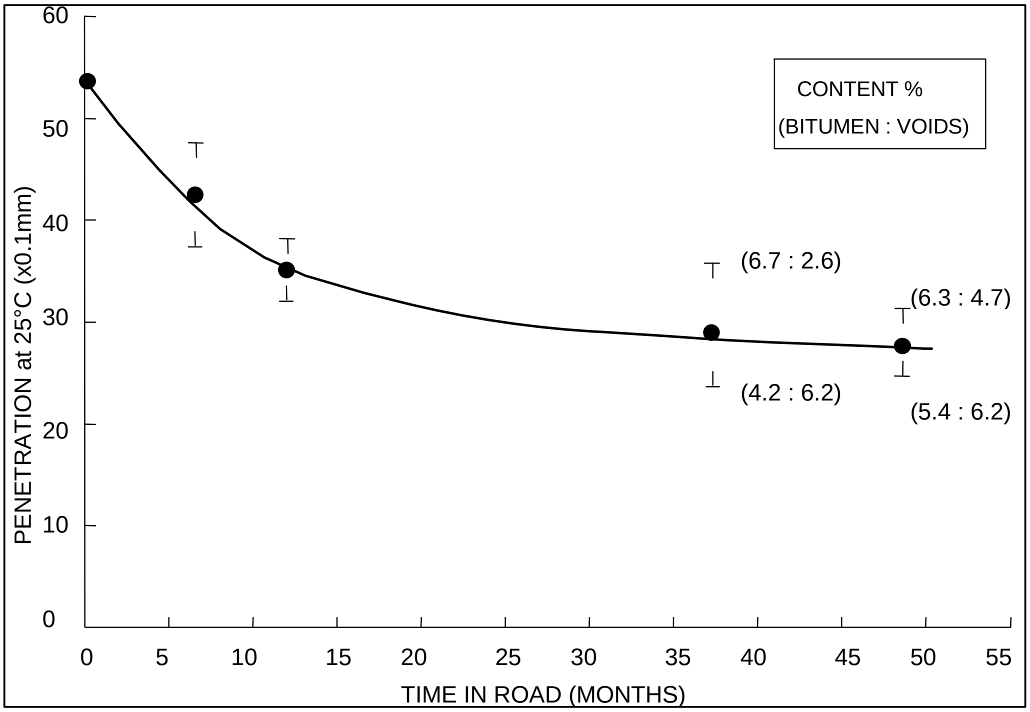


Figure E.2 Relationship between age and bitumen penetration for surface dressed bitumen macadam roadbase.

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 TRH3, 1986) 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) can also be used to surface the HMA layer.

E.4 Transfer of refusal density mix design to compaction trials

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. The will ensure that the densities of cores and loose material compacted in the laboratory can be properly compared.

A minimum of three trial lengths should be constructed with bitumen contents at the laboratory optimum (see 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.

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.

A minimum of 93% and a mean value of 95% of refusal density is 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 F: PERFORMANCE TEST FOR HMA

F.1 Introduction

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. 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. Although the Marshall design method addresses these problems, it has been found that the correlation between measurements of stability and flow and subsequent performance in the road (Whiteoak, 1990) is not as good as is desired. 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 tend to be unreliable.

Additional tests are therefore particularly desirable 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; and
- (iii) wheel tracking.

Performance tests do not in fact guarantee the long term performance of an HMA because the tests are carried out on laboratory prepared samples which have not been subjected to traffic loading or to environmental ageing. However, they do have considerable potential for identifying mixes that would be unsuitable for use and with increasing experience of their application in specific environments they can be expected to become standard procedures in the future.

Different test conditions and requirements are specified in different countries and, therefore, it is difficult to develop generalised specifications for these tests. The following paragraphs describe the test methods and the alternative specifications that are used. Users are advised to refer to those specifications that are most relevant to the local conditions of traffic, climate and aggregate type as a starting point for developing their own specifications.

F.2 Mix Stiffness Modulus (or Resilient Modulus)

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

F.2.1 Indirect tensile stiffness

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.

Different test conditions and performance specifications are recommended by authorities in several countries. The main differences are summarised in Tables F.1 to F.3.

Table F.1 Differences between test methods

Difference	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 Poisson ratio	0.35	0.35	0.4

F.2.2 Specifications for indirect tensile stiffness

Current specifications based on the different performance tests are given below and most are still under development. Authorities should continue further studies to develop specifications that are suitable for their own local conditions and materials.

The UK specifications shown in Table F.2 apply to a macadam material used for the roadbase and binder course layers with a void content of between 7-9%.

Table F.2 UK 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

The Australian specification 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. The criteria refer to a mix containing 4-5% air voids and are shown in Table F.3 below.

Table F.3 Australian Specification for Mix Stiffness Modulus

Bitumen class	Viscosity (Pa.s)	Maximum particle size (mm)			
		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

F.3 Creep Stiffness Modulus

Creep stiffness modulus is defined as a function of the stress (applied load) and strain (deformation) and can be determined by dynamic or static tests on core samples under unconfined uniaxial loading. The intended purpose of the test is to provide a means of ranking bituminous mixes, in terms of their likely performance with respect to deformation under traffic.

(NB: The present UK standard is a static test (BS 598: Part 111:1995) but this will be superseded by a dynamic test procedure, referred to as the RLAT (Repeated Load Axial Test)).

Dynamic testing is also recommended in American and Australian standards.

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 F.4.

Table F.4 Differences between test methods

Test details	AUSTRALIA	S AFRICA	UK	US
Reference Document	AS 2891.12.1-1995	TMH 1	BS DD 226:1996	ASTM D3497 1999
Number of specimens required for testing	3	3	6	3 *
Core diameter (mm)	100 or 150	101.6±2 for cores	100, 150 *or* 200	4 x max stone size
Core depth (mm)	50 or 75	50 or 2.5 x max stone size	<100	2 x core diameter
Test temperature (°C)	50	40±1	30	5, 25 and 40
Stress load (kPa)	200	200	100	Variable up to 240
Pulse load frequency (Hz)	2	Static load	2	1 and 4 and 16
Stress applications	Max of 40,000	-	1800	30-45s duration for each load

Note 6 cores are required when taken from a pavement.

Creep modulus can be calculated for any given number of stress applications and this also results in differences in the modulus quoted by each authority. In summary the major differences are;

(i) In the UK, 1800 load applications are made at a test temperature of 30°C.

(ii) ASTM requires the value to be calculated at three temperatures and three loading frequencies within a loading time of between 30 and 45 seconds.

(iii) The Australian standard requires the accumulated strain to be calculated for each recorded cycle up to 10,000, or if necessary, 30000 micro-strains. This data is processed to give the minimum rate of increase of accumulated strain, referred to as minimum slope, against load cycles.

(iv) The South African procedure is a static load test and the creep is measured at 1 minute intervals up to 10 minutes and then every 10 minutes up to a maximum of 100 minutes. As well as allowing the creep modulus to be calculated, for each time interval deformation is expressed as a percent change from zero divided by the average height of the specimen. These values are plotted against log time from which help to characterise the behaviour of the mix.

(NB: Test procedures for this constrained creep test (VRLAT) are presently being developed in the UK.)

F.3.1 Creep Modulus Specifications

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 specification and the proposals are shown in Table F.5.

Table F.5 UK specification for Creep stiffness of DBM roadbase and binder course layers

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

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

The Australian specification, shown in Table F.6 below, considers traffic levels as well as pavement temperature and the criteria is based on minimum creep slope values. The traffic categories referred to in Table F.6 are described in Table F.7.

Table F.6 Australian Creep stiffness specification [rate of accumulated strain against load cycles]

Temperature WMAPT (°C)*	Sample VIM (%)	Traffic		
		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

* weighted mean annual pavement temperature

Table F.7 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

The South African specification shown in Table F.8 refers to air void contents of between 3 to 6 per cent and 2 to 5 per cent depending on the traffic category.

Table F.8 South African Creep Specification

Traffic Category (esa/lane)	Air voids (%)	Creep modulus (MPa)
$12 – 50 \times 10^6$	3 – 6	80
$3 – 12 \times 10^6$	3 – 6	60
$0.8 – 3 \times 10^6$	2 – 5	35

F.4 Wheel-Tracking Test

The wheel-tracking test is a British Standard test (BSI 598:Part 110: 1998). A test 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 appears to be relatively insensitive when used to test continuously graded materials, such as asphalt concrete, at a temperature of 45°C.

Samples can be 200mm diameter cores or laboratory-prepared slabs compacted in a standard purpose-made mould. Thus the samples can be obtained in the following ways;

(i) compacted in a standard rectangular steel mould with a pedestrian compaction roller and tested as a slab or as cores cut from the slab;

(ii) cores cut from road sites; the cores can be sawn to the specified depth with a diamond cutting blade.

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 whether as slabs or cores cut from slabs.

F.4.1 Specifications for performance tests

Current UK specifications are given in Table F.9. These are a useful first approximation, but authorities must develop specifications suitable for local conditions and materials. In many countries which experience high pavement temperatures a test temperature of 60°C will be most appropriate.

Table F.9 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 G: Recycling of Bituminous Materials

G.1 Introduction

The use of thick bituminous surfacings 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 areas where there is a shortage of road-building aggregate or where there are no indigenous oil reserves.

It is possible to ensure that there are benefits both for client and contractor from recycling operations but recycling is unlikely to become widespread 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.

In tropical countries the types of asphalt concrete 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 is brittle, containing 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.

G.2 Methods of recycling

Reclaimed Asphalt Pavement (RAP), or millings, is recycled in three main 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. The greatest cost savings will be obtained when RAP is used to produce good quality bitumen bound material and its use as unbound material should be regarded as the minimum target for recycling.

The decision to recycle asphalt 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.

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 layers 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.

When RAP is to be used in a pavement layer, good quality control of the RAP stockpiles will be vital for the manufacture of consistent HMA. This may require a considerable amount of testing. The presence of old multiple surface dressings may also 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.

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.

G.3 Suggested method of sampling existing asphalt

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 can be manufactured with the available plant. 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) wheel paths 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.

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 recovered bitumen of less than 20) from less hardened materials and to discard badly contaminated material from ‘oil lanes’. Detailed assessment of stockpile management should be finalised after a desk study has been carried out to show how the various RAP materials can be combined with fresh aggregate to produce acceptable mixes.

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

i) particle size distribution;
ii) bitumen content;
iii) viscosity of recovered bitumen; and
iv) an assessment of crushability.

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.

G.4 Methods of obtaining RAP

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.

G.4.1 Asphalt millings

Asphalt millings are obtained by planning, 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.

G.4.2 Crushed asphalt

Crushed asphalt is commonly obtained by using horizontal impact crushers or hammer mill 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.

G.4.3 Granulated asphalt

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

G.5 Stockpiling RAP

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.

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 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.

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.

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.

G.6 Use of RAP as unbound granular material

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 re-compact in the new layer. For example, bitumen in RAP with a penetration value of less than about 15 will behave in a brittle manner.

In contrast, asphalt that 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 wheel paths, 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 re-blending and adding fresh aggregate in a purpose-made hot-mix recycling plant.

G.6.1 Outline of UK specification for use of RAP as capping layer

A capping layer would only be used in the construction of a new pavement where the *in situ* subgrade CBR is less than 5 per cent.

The UK specification requires milled or granular RAP to meet the grading requirements given in Table G.1. 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.

Table G.1 Grading requirements for RAP for use in capping layers

BS sieve size (mm)	Percent passing sieve size
125	100
90	80-100
75	65-100
37.5	45-100
10	15-60
5	10-45
0.6	0-25
0.063	0-12

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), is a suitable specification.

Other unbound granular material can be added to the RAP to give a material with reduced effective bitumen content.

A higher quality material can be obtained by limiting the maximum effective particle size to 37.5mm by screening out and re-crushing oversized material.

G.6.2 Outline of UK specification for use of RAP as sub-base

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.

In the UK, RAP for sub-base should conform to the particle size distribution given in Table G.2.

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 mould, a loading frame and a vibrating hammer.

Table G.2 Range of lump-size grading of RAP for use in sub-base

BS sieve size (mm)	Percent passing sieve size
75	100
37.5	85-100
20	60-90
10	30-70
5	15-45
0.6	0-22
0.075	0-10

Notes

1 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 2)

2. The planning 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.

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. Adoption the Trafficking Trial procedure could help to achieve this.

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

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 wheel paths is measured. For the material to be approved the mean deformation must be less than 30mm.

G.6.3 Use of RAP as granular roadbase

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.

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 provides an appropriate method of acquiring this experience.

G.7 Cold mix recycling

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 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. Cold mix recycling is outside the scope of this manual. Reference should be made to appropriate manuals (see Bibliography) for detailed recommendations.

G.8 Plant hot mix recycling

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.

G.8.1 RAP feed to plant

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

i) The sides should be steeper than those of an aggregate feed bin.
ii) The bottom of the bin may be longer and wider than that of an aggregate feed bin.
iii) The bottom of the bin may slope downwards, to match an angled feed belt, and the end wall is sometimes left open.
iv) Vibrators should not be used.
v) RAP should be delivered slowly into the cold feed bin from the front-end loader.
vi) 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.
vii) 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 later.

G.8.2 Batch plant recycling

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

i) The temperature to which the virgin aggregate is heated.
ii) The temperature and moisture content of the RAP.
iii) 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.

G.8.3 Batch mixers with a separate heating drum (parallel drum)

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.

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.

G.9 Evaluation and design - plant hot-mix recycling

G.9.1 Variability of RAP

RAP will usually be either 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.

G.10 Bitumen rejuvenators

Rejuvenators have been used to change the properties of bitumen in RAP to make it 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)).

G.11 Blending with a soft bitumen

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 G.1 (Whiteoak, 1990):

$$LogP = (A.\log P_a + B.\log P_b)/100 \qquad \text{Equation G.1}$$

where: P = specified penetration of final blend.

P_a = penetration of RAP bitumen.

P_b = 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.

G.11.1 Limitations of bitumen blending

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.

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.

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.

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.

G.12 Mix design

The most common design procedure 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 8 of this manual. These recycled materials must be sealed or surfaced with a new bituminous wearing course.

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.

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 Appendices C and E can be adopted and the resultant mix evaluated in field trials.

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 E). The Percentage Refusal Density Test should be used to check the density of the laid material.

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 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.

G.13 Recycling feasibility case studies

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 G.3. Site details are given in Table G.4.

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.

Table G.3 Locations of core sampling

Chainage	Between verge side wheel path and road edge	Verge side wheel path	Centre line of lane	Off side wheel path
0	✓	✓	✓	✓
100		✓		
200	✓	✓	✓	✓
300		✓		
400	✓	✓	✓	✓
500		✓		
600	✓	✓	✓	✓
700		✓		
800	✓	✓	✓	✓
900		✓		
1000	✓	✓	✓	✓

Table G.4 Details of road sites

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

G.13.1 Testing of the core samples

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.

G.13.2 Case study 1

Analyses of core samples are shown in Table G.5. The mean bitumen content and aggregate grading for the wearing course and binder course were significantly different, as would be expected, and the variability of bitumen content and penetration within each layer was low.

In practice the wearing course and binder course could be stockpiled separately or as a combined 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.

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

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 G.2 for sub-base.

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.

Table G.5 Summary of layer composition: Case Study 1

BS sieve (mm)	Per cent passing sieve size			
	Wearing course		Binder course	
	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	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	4.9-5.7	3.6	3.1-4.4
Penetration (0.1mm)	13	6-24	9	5-15

Hot mix recycling

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.

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.

As indicated in Table G.5, 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.

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 G.6. 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.

Table G.6 An example of blending fresh aggregate and RAP: Case Study 1

Sieve (mm)	Superpave particle size distribution limits			Blend 40% RAP and fresh aggregate
	Control points		Restricted zone	
	Min	Max		
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

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.

The results of the performance tests summarised in Table G.7 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.

Table G.7 Laboratory performance test results for a recycled mix: Case Study 1

No. of samples	VIM (%)	% of refusal density	Wheel tracking rate (mm/hr at 60^0C)	ITT (GPa)	
				At 20^0C	At 30^0C
6 per test	6.5 - 7.3	95.3 - 96.0	0.17 - 0.42	5.7 - 8.0	2.5 - 3.6

G.13.3 Case Study 2

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 G.8.

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 G.9 and G.10.

Table G.8 Thickness of cores: Case Study 2

Chainage	Core Nos.	Core thicknesses			
		Near road edge	Verge side wheel path	Centre of lane	Off-side
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 G.9 VIM and bitumen content: Case Study 2

Layer	Height above roadbase (mm)	No. of cores analysed	No. with VIM < 3%	Bitumen content (5)		
				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	16	10	4.3	0.4	3.7-5.0

Table G.10 Aggregate particle size distribution: Case Study 2

BS sieve (mm)	Per cent passing sieve size					
	Layer 1		Layer 2		Layer 3	
	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

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 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 G.11.

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.

Table G.11 Penetration of RAP bitumen & in blend with fresh bitumen: Case Study 2

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

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

It is 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 80/100 penetration grade bitumen should ensure that mixing with new aggregate would produce a mix with acceptable uniformity.

A laboratory design for a roadbase mix was carried out using fresh aggregates. A blend of 50 per cent RAP and fresh aggregate was designed to conform with the Asphalt Institute requirements for a mix having a nominal maximum aggregate size of 25mm as shown in Table G.12.

The design bitumen content was found to be 3.5 per cent of which 1.3 per cent was 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 refusal density, for Indirect Tensile Tests (ITT) and wheel tracking tests.

The results of the performance tests on the recycled mix are shown in Table G.13. Wheel tracking rates were low and well within the UK specification for very heavily stressed sites (Appendix F) whilst the ITT results were marginal for a mix containing 60/70 penetration grade bitumen. It is considered that the wheel tracking test gives the better indication of stability and it was concluded that the mix should be stable under traffic.

Table G.12 An example of blending fresh aggregate and RAP: Case Study 2

Sieve size (mm)	Passing sieve size (%)	
	Asphalt Institute grading 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
0.3	5-17	12
0.075	1-7	6

Table G.13 Laboratory performance test results for a recycled mix: Case Study 2

No. of samples	VIM (%)	% of refusal density	Wheel tracking rate (mm/hr at 60^0C)	ITT (GPa)	
				At 20^0C	At 30^0C
6 per test	7.0	96.0	0.12-0.46	-	1 - 2

Appendix H: THEORETICAL AND EMPIRICAL BASIS FOR THE DESIGN CHARTS

H.1 Mechanistic or analytic methods

Mechanistic methods are based on a more detailed understanding of the behaviour of the materials in the road pavement. The principle is that a suitable theory is used to calculate the stresses and strains that occur within the pavement as a result of the external loading (traffic). The way that the materials respond to these stresses is then calculated based on knowledge obtained from studies in which the materials have been subjected to similar stresses and strains in the laboratory and their behaviour expressed in terms of suitable equations (models). This latter process is essentially empirical; there is nothing fundamental in the equations used to describe the response of the materials to the imposed stresses. The method is often referred to as a more fundamental method simply because it is based on knowledge of the basic stresses and strains in the pavement. Therefore, at least in principle, many different materials can be tested in the laboratory under a range of conditions, and then used successfully in the road pavement, thereby eliminating the need for lengthy and relatively limited evaluation using full-scale empirical trials.

The performance criteria are usually ‘fatigue’ type relationships linking the value of the stress or strain with the number of times that the stress or strain can be repeated before ‘failure’ occurs. A great deal of good laboratory-based research has been done on the fatigue laws governing bitumen-bound materials, especially hot mix asphalt (HMA). In contrast, very little has been done on the fatigue relationships for subgrade materials. This is an important omission.

Sometimes critical points occur in other pavement layers but less research has been done to quantify the criteria that should be used. This is because the behaviour of these layers is usually controlled by requiring the materials to meet minimum, and relatively safe, specifications. These are usually simple pass/fail tests, such as CBR tests.

There are several important and outstanding problems with mechanistic methods that need to be understood and respected.

First, a suitable theory is needed that accurately predicts the stresses in the pavement. Since the materials are complex, most theories are inaccurate.

Secondly, and just as importantly, the laboratory testing cannot simulate the conditions in the real road adequately and so the equations that predict the response of the materials to the stresses are also inaccurate. For example, one serious problem is that the properties of the materials change over the course of time and laboratory experiments cannot deal with this.

Thirdly, the response of the materials in the laboratory is not the same as in the road because the road is a more complex structure and is loaded in a more complex manner. Translating laboratory behaviour into road behaviour is not an easy task and some assumptions have to be made.

The inadequacy of pavement failure criteria is also recognised as an important shortcoming in our current knowledge. One of the key critical stresses/strains is the vertical stress at the top of the subgrade. Several alternative equations developed by different researchers are used to define the criterion but, whichever one is chosen, it is usually applied to *all* subgrade materials. It is illogical to assume that all subgrade materials are governed by the same failure criterion (i.e. the relationship between strain and the number of allowable strain repetitions) but this is inevitably what is assumed in most methods of analysis. The subgrade strain criterion has been investigated (Janoo *et al*, 2003; Cortez, 2007) and showed that subgrade strain criterion can differ by several orders of magnitude between materials and between samples of the same material at different density and moisture conditions. The significance of this cannot be overstated but the criterion usually used in theoretical analysis is based on weak subgrades, which results in very safe but often over-designed structures.

Although the correlation between elastic modulus (E) and 'strength' is very poor (usually much too poor for use in design calculations) the simple relationship,

$$E = a*CBR$$

between E and the California Bearing Ratio (CBR, a simple measure of strength not modulus) is almost universally used to estimate the elastic modulus especially of subgrade material. This leads to significant inaccuracies.

Fatigue in bituminous materials has been much researched in the laboratory. The major problem that arises is that laboratory conditions are quite different to field conditions. The accelerated testing in the laboratory prevents any visco-elastic 'healing' from occurring and, as a consequence, fatigue lives measured in the laboratory can be shorter than those in the field by several orders of magnitude. Coupled with ageing of the surfacing and the widespread occurrence of top-down cracking that generally precedes the traditional form of fatigue failure in HMA, serious problems still remain concerning performance prediction.

In view of all the problems associated with the mechanistic method it is perhaps surprising that the method is used so frequently with apparent success. There are several reasons for this. First of all the 'errors' in the mechanistic method itself are systematic rather than random. This means that the relative differences in the results for different pavements are much more meaningful than the absolute values. If the method is calibrated by comparison with measured performance, then sensible conclusions may be drawn. This assumption requires that the calibration is 'correct' and this means that the performance of the pavements used for calibration must be thoroughly understood (e.g. the origin of cracks must be known with certainty).

Secondly, most pavements will not differ much from those that should have been used to calibrate the mechanistic model. For example, roads with granular unbound bases and structural HMA surfacings will normally consist of an unbound sub-base and road base of materials meeting standard specifications and of thickness ranging from about 250 mm up to 500 mm. The thickness of the HMA surfacing will be between, say, 100 mm and 250 mm. Thus, provided the mechanistic model has been calibrated for the specific type of structure under consideration, the model stands a good chance of predicting the relative performance of similar pavements with reasonable accuracy. However, it must be emphasised that this *does* require a good calibration in which the real cause or causes of failure in the roads used for calibration are thoroughly understood.

H.2 Design criteria

The asphalt fatigue criterion developed in Australia is recommended because it is based on an evolving research programme and is, therefore, based on the best available data. Furthermore, the climatic conditions in Australia are suitably tropical.

The fatigue law for asphalt is;

$$N = \left[\frac{6918 \times (0.856 \times V_b + 1.08)}{\mu\varepsilon \times S_{mix}^{0.36}}\right]^5$$

where:

- V_b = proportion of bitumen by volume in the mixture in %
- S_{mix} = elastic modulus of mixture in MN/m^2
- $\mu\varepsilon$ = *horizontal* microstrain in the asphalt
- N = number of strain repetitions to failure.

It can be seen that the use of these equations requires knowledge of the elastic modulus of the asphaltic concrete and the volume of bitumen it contains. Furthermore, the predicted 'life' of the asphalt is extremely sensitive to the value of both of these factors and both are sufficiently variable to ensure that sufficiently accurate predictions of 'life' are extremely difficult to make, hence the need for local calibration. Alternatively, assumptions can be made that will ensure a sufficient factor of safety.

The subgrade strain criterion developed in Australia is similar to the criterion developed from the performance of the pavements on the stronger subgrades in some recent research in a pavement test facility in the USA (Janoo *et al*, 2003) but much less conservative than the much older but frequently used criteria that are based on more limited data. The criteria developed by Shell are also conservative but conveniently include equations for different levels of reliability.

The Australian criterion is

$$N = \left(\frac{9300}{\mu\varepsilon}\right)^7$$

The recent study in the USA yields an *average* criterion of

$$N = \left(\frac{11000}{\mu\varepsilon}\right)^{5.7}$$

The Shell 50% criterion is

$$N = \left(\frac{28{,}000}{\mu\varepsilon}\right)^4$$

It is also noticeable that the slope of the criterion lines developed by different authors also varies considerably. Thus the precision with which subgrade criteria are known is poor and the range of the published criteria is very wide indeed. Selecting the most appropriate is essentially a matter of engineering judgement.

The implied subgrade strain criterion that can be derived from the empirically-based design charts in Overseas Road Note 31 is similar to the Shell criterion at only 50% reliability but more conservative by a factor of about three (in traffic terms) than the *average* criteria derived from the study in the USA and by a factor of about 10 compared with the Australian criteria. This criterion is suggested as a suitable compromise until better criteria can be developed based on measured properties of the subgrade material.

The criterion is

$$N = \left(\frac{7300}{\mu\varepsilon}\right)^{6.2}$$

where: $\mu\varepsilon$ = *vertical* microstrain in the subgrade.

Table H.1 summarises the critical levels of strain for the asphalt and the subgrade for each traffic class using the criteria described above.

Table H.1 Critical strains

Traffic	**T3**	**T4**	**T5**	**T6**	**T7**	**T8**	**T9**	**T10**
	1.5	**1.5 – 3.0**	**3 – 6**	**6 – 10**	**10 – 17**	**17 - 30**	**30 - 50**	**50 - 80**
HMA microstrain criteria								
Volume of Binder								
11 %		205	180	160	145	130	117	107
10.5 %		198	172	155	140	125	113	103
9.5 %		181	157	142	128	114	103	94
Subgrade microstrain criteria								
	805	720	645	595	545	495	460	425

In this analysis the following values shown in Table H.2 have been used.

Table H.2. Material characteristics for mechanistic analysis

Material	Parameter	Value	Comment
Asphaltic concrete wearing course and binder course	Elastic modulus (MPa)	3000	A balance between a value appropriate for high ambient temperatures and the effect of ageing and embrittlement
	Volume of bitumen	10.5%	
Asphaltic concrete roadbase	Elastic modulus (MPa)	3000	
	Volume of bitumen	9.5%	
Granular roadbase	Elastic modulus (MPa) Poisson's ratio	300 0.30	For all qualities with CBR > 80%
Granular sub-base	Elastic modulus (MPa) Poisson's ratio	175 0.30	For CBR ≥30%
Capping layer	Elastic modulus (MPa) Poisson's ratio	100 0.30	For CBR ≥15%
Subgrades S1 S2 S3 S4 S5 S6	Elastic modulus in MPa	 28 37 53 73 112 175	Poisson's ratio for all subgrades was assumed to be 0.4
Hydraulically stabilised material	Elastic modulus (MPa)	CB1 = 3500 CB2 = 2500 CS =1500	Poissons ratio assumed to be 0.25 The modulus of CS is assumed to decrease with time hence a conservative low value of 1000MPa has been used

H.3 Characteristics of each Design Chart.

The catalogue includes designs for very weak subgrades (S1) but such subgrades are very difficult to deal with and it is likely that a special investigation will be needed to determine the best solution.

In all of the charts, the basic granular sub-base of GS quality material and the hydraulically stabilised sub-base material CS are interchangeable.

H.3.1 Chart A1

Charts A1 is for pavements with a surface treatment comprising a double surface dressing or an Otta seal on top of a granular road base. The sub-base is either a granular material or a weakly cemented hydraulically bound material. The thickness of the overall structure is controlled by the need to limit the strain in the subgrade and care must be taken to ensure that the roadbase is high quality for the higher traffic levels as indicated.

Table H.3 shows the computed subgrade strains in comparison with the criteria. It will be noted that, for the weaker subgrades, the criteria are more easily met. For each traffic level it is recognised that a single subgrade criterion for all subgrades cannot be correct and that weaker subgrades require more conservative criteria. The thicknesses of the pavement layers in this Chart provide a degree of safety for the weaker subgrade classes.

Table H.3 Strain in the subgrade for Chart A

Subgrade class	T3	T4	T5	T6
	1.5 mesa	**1.5 – 3.0 mesa**	**3 – 6 mesa**	**6 – 10 mesa**
Strain criterion	**805**	**720**	**645**	**595**
S1	610	540	480	425
S2	715	625	545	480
S3	770	665	575	530
S4	765	705	650	555
S5	745	680	630	575
S6	700	665	665	625

H.3.2 Chart A2

Similar to Chart A1 but this chart makes use of the benefits of a hydraulically stabilised lower roadbase. Such structures have performed well and are generally considered more reliable than their unbound granular counterparts. This is partly because the cemented layer provides a good foundation for compacting the layers above and because it is also more tolerant of water. The subgrade strain is low with such a structure but, nevertheless, a capping layer is required to support the cemented layer and for carrying construction traffic.

H.3.3 Chart A3

Chart A3 shows designs using a hydraulically stabilised roadbase. Such a roadbase is strong but will contain fine cracks resulting from the curing and shrinkage of the cemented material. These cracks do not seriously affect the traffic carrying capacity of the pavement but they will eventually appear through the surface treatment and will need to be sealed with a maintenance reseal at some stage before further deterioration can occur. The cement stabilised roadbase and sub-base are efficient load spreading layers and subgrade strains will not exceed the critical values.

H.3.4 Chart B

Chart B shows designs utilising a thin HMA surfacing. The structures themselves are similar to those in Chart A because thin surfacings add very little to the overall structural strength. A flexible HMA surfacing (see Chapter 8) is suitable for the lower traffic levels but experience in the Republic of South Africa has also shown that a continuously graded HMA surfacing (e.g. the nominal 9.5mm maximum stone size in Table 8.7) also works well and is the favoured option in RSA. No evidence of ‘bottom-up’ fatigue cracking has been reported but it should be noted that construction and mix design must be carried out to high standards. The importance of providing a very stiff supporting structure beneath the thin continuously graded asphalt concrete cannot be over emphasised. Thus a strong aggregate roadbase is required and a stabilised sub-base is preferred

H.3.5 Chart C1.

Chart C is a common design using a high quality structural surfacing of asphaltic concrete (wearing course and binder course) on a granular roadbase. Such a structure is designed so that the strain in the asphalt will not cause fatigue failure. Unfortunately the strain is most sensitive to the thickness and stiffness of the asphaltic concrete itself and relatively insensitive to the thickness of the unbound layers below, hence, if the design is to follow the mechanistic design method, the asphalt layer must be quite thick for high traffic levels.

H.3.6 Chart C2

This is similar to Chart C1 but using a hydraulically bound lower roadbase layer. The modulus of such a layer is considerably greater than that of an unbound layer and this enables the strain in the asphaltic concrete to be reduced substantially. The upper roadbase of granular material prevents any shrinkage cracks in the cemented lower roadbase from causing sympathetic cracking in the asphaltic concrete.

H.3.7 Chart D

This is another traditional design that also reflects the difficulty of preventing fatigue failure in the asphaltic concrete. In this solution, sometimes called a full-depth AC pavement, the roadbase is a bitumen-stabilised layer and the surfacing is also an asphaltic concrete. However, in tropical areas and for heavy traffic, the bituminous roadbase must be of relatively high specification (Pavement Design Manual – 2011, Chapter 8). The thickness of asphaltic material for the higher traffic levels is in the range that is considered to be ‘long-life’. In other words, no fatigue failure is ever likely to occur; all cracking will be ‘top-down’ and rehabilitation should consist solely of milling off the top 30 - 50mm of aged and brittle material and replacing it.

H.4 The Structural Number approach.

The concept of structural number was first introduced as a result of the AASHO Road Test as a measure of overall pavement strength. It is essentially a measure of the total thickness of the road pavement weighted according to the ‘strength’ of each layer and calculated as follows:

$$SN = 0.0394 \sum a_i.h_i$$

where:

SN = structural number of the pavement,
a_i = strength coefficient of the i^{th} layer,
h_i = thickness of the i^{th} layer, in millimetres,
and the summation is over the number of pavement layers, n.

The individual layer strength coefficients are determined from the normal tests that are used to define the strength of the material in question e.g. CBR for granular materials, UCS for cemented materials etc. Table H.4 shows typical values.

The coefficients can be modified to take into account the deterioration or weakening of the materials caused by environmental effects, for example, high moisture contents in unbound materials caused by poor drainage and high temperature conditions affecting bituminous materials.

Table H.4 Pavement layer strength coefficients

Layer	Layer Type	Condition	Coefficient
Surfacing	Surface dressing		$a_i = 0.1$
	New asphalt concrete wearing [1,2]	MR_{30} = 1500 MPa	$a_i = 0.30$
		MR_{30} = 2000 MPa	$a_i = 0.35$
		MR_{30} = 2500 MPa	$a_i = 0.40$
		$MR_{30} \geq$ 3000 MPa	$a_i = 0.45$
Road base	Asphalt concrete	As above	As above
	Granular unbound	Default	$a_i = (29.14\ CBR - 0.1977\ CBR^2 + 0.00045\ CBR^3)\ 10^{-4}$
		GB 1 (CBR > 100%)	0.145
		GB 2 (CBR = 100%)	0.14
		GB 3 (CBR = 80%)	
		With a stabilised layer underneath	0.135
		With an unbound granular layer underneath	0.13
		GB 4 (CBR = 65%) [4]	0.12
		GB 5 (CBR = 55%)[4]	0.107
		GB 6 (CBR = 45%)[4]	0.01
	Bitumen treated gravels and sands	Marshall stability = 2.5 MN	a = 0.135
		Marshall stability = 5.0 MN	a = 0.185
		Marshall stability = 7.5 MN	a = 0.23
	Cemented[3]	Equation	$a_i = 0.075 + 0.039\ UCS - 0.00088(UCS)^2$
		CB 1 (UCS = 3.0 – 6.0 MPa)	a = 0.18
		CB 2 (UCS = 1.5 – 3.0 MPa)	a = 0.13
Sub-base	Granular unbound	Equation	$a_j = -0.075 + 0.184(\log_{10} CBR) - 0.0444(\log_{10} CBR)^2$
		GS (CBR = 30%)	a = 0.105
		GC (CBR = 15%)	a = 0.08
	Cemented	CB 3 (UCS = 0.7 – 1.5 MPa)	a = 0.1

Notes:

1. See discussion above.
2. Unconfined Compressive Strength (UCS) is quoted in MPa at 14 days.
3. MR_{30} is the resilient modulus by the indirect tensile test at 30 °C.
4. Used for low volume roads (see LVR manual).

H.4.1 Modified Structural Number

The AASHO Road Test was constructed on a single subgrade, therefore the effect of different subgrades could not be estimated and the structural number could not include a subgrade contribution. To overcome this problem and to extend the concept to all subgrades, a subgrade contribution was derived as described by Hodges et al. (1975) and a modified structural number defined as follows:

$$SNC = SN + 3.51\ (\log_{10} CBR_s) - 0.85\ (\log_{10} CBR_s)^2 - 1.43$$

where:

SNC = Modified structural number of the pavement

CBR = in-situ CBR of the subgrade

The modified structural number (SNC) has been used extensively and forms the basis for defining pavement strength in many pavement performance models.

H.4.2 Adjusted Structural Number

When evaluating a pavement in order to design rehabilitation measures, it is found that many pavements cannot be divided easily into distinct roadbase and sub-base layers with a well-defined and uniform subgrade. Hence, when calculating the structural number according to the equation above, the engineer has to judge which layers to define as roadbase, which as sub-base, and where to define the top of the subgrade. For many roads this has proven quite difficult. There are often several layers that could be considered either as sub-bases or part of the subgrade, especially where capping layers or selected fill have been used. The simple summation over all the apparent layers allows the engineer to obtain almost any value of structural number since the value will depend on where the engineer assumes that the sub-base(s) end and the subgrade begins. In the past this problem has been addressed by simply limiting the total depth of all the layers that are considered to be road pavement. However, this is somewhat arbitrary, has not been used universally, and has led to unacceptably large errors in some circumstances.

The problem arises because the contributions of each layer to the structural number are independent of depth. This cannot be correct since logic dictates that a layer that lies very deep within the subgrade can have little or no influence on the performance of the road. To eliminate the problem, a method of calculating the modified structural number has been devised in which the contributions of each layer to the overall structural number decrease with depth (Rolt and Parkman, 2000).

To distinguish the structural number derived from the original Modified Structural Number (SNC), the new structural number is called the Adjusted Structural Number (SNP). It is calculated as follows:

$$SNP = SNA + SNS + SNG$$

Where the component terms are calculated follows;

$$SNA = 0.0394 \sum_{i=1}^{n} a_i\, h_i$$

$$SNS = 0.0394 \sum_{j=1}^{m} a_j \left\{ \left(\frac{b_0 \exp(-b_3 z_j)}{-b_3} + \frac{b_1 \exp(-(b_2 + b_3) z_j)}{(b_2 + b_3)} \right) - \left(\frac{b_0 \exp(-b_3 z_{j-1})}{-b_3} + \frac{b_1 \exp(-(b_2 + b_3) z_{j-1})}{(b_2 + b_3)} \right) \right\}$$

$$SNG = (b_0 - b_1 \exp(-b_2 z_m))\,(\exp(-b_3 z_m))\,[3.51 \log_{10} CBR_s - 0.85(\log_{10} CBR_s)^2 - 1.43]$$

and

SNP	=	adjusted structural number of the pavement
SNA	=	contribution of surfacing and base layers
SNS	=	contribution of the sub-base and selected fill layers
SNG	=	contribution of the subgrade
n	=	number of base and surfacing layers (i = 1, n)
a_i	=	layer coefficient for base or surfacing layer i
h_i	=	thickness of base or surfacing layer i, in mm
m	=	number of sub-base and selected fill layers (j = 1, m)
a_j	=	layer coefficient for sub-base or selected fill layer j for season s
z	=	depth parameter measured from the top of the sub-base (underside of base), in mm
z_j	=	depth to the underside of the jth layer ($z_0 = 0$), in mm
CBR	=	in situ subgrade CBR

The values of the model coefficients b_0 to b_3 are given in Table H.5.

Table H.5 Adjusted structural number model coefficients

b_0	b_1	b_2	b_3
1.6	0.6	0.008	0.00207

It should be noted that for roads that have been built according to the designs in the ERA *Pavement Design Manual -2011*(or any other manual for that matter) with well defined layers of uniform strength, the Adjusted Structural Number and the Modified Structural Number are essentially identical. The value of SNP is calculated when evaluating a pavement with many layers of varying strength.

H.4.3 Structural Numbers of the catalogue designs

Tables H.6 and H.7 show the values of SN and SNC for different subgrade conditions and design traffic levels for the various pavement types calculated from the design charts in the ERA *Pavement Design Manual.*

Table H.6 Structural Numbers for Designed Structures

Chart No	Subgrade	T1 <0.3	T2 0.3 - 0.7	T3 1.0 - 1.5	T4 1.5 - 3.0	T5 3 - 6	T6 6 - 10	T7 10 - 17	T8 17 - 30
Chart A1	S1	2.38	2.56	2.89	3.10	3.38	3.63		
	S2	2.04	2.27	2.47	2.68	2.97	3.21		
	S3	1.78	1.91	2.03	2.26	2.54	2.78		
	S4	1.59	1.72	1.82	2.05	2.23	2.47		
	S5	1.38	1.49	1.61	1.74	1.96	2.20		
	S6	0.87	0.99	1.25	1.38	1.48	1.61		
Chart A2	S1	2.58	2.71	2.83	2.96	3.28	3.43	3.55	
	S2	2.14	2.26	2.39	2.52	2.83	2.98	3.24	
	S3	1.90	1.90	2.03	2.23	2.55	2.82	2.95	
	S4	1.51	1.63	1.76	1.89	2.20	2.35	2.61	
	S5	1.38	1.38	1.51	1.63	1.82	1.95	2.20	
	S6	0.87	0.87	0.99	1.12	1.34	1.56	1.82	
Chart A3	S1	2.48	2.58	2.71	2.83	3.03	3.13	3.26	
	S2	2.17	2.26	2.39	2.52	2.72	2.91	3.04	
	S3	1.93	1.93	2.06	2.16	2.36	2.56	2.69	
	S4	1.46	1.56	1.68	1.88	2.11	2.30	2.43	
	S5	1.26	1.26	1.39	1.58	1.81	1.91	2.04	
	S6	0.87	0.87	0.99	1.12	1.25	1.38	1.51	

Table H.6 (continued). Structural Numbers for Designed Structures

Chart No	Subgrade	T3 1.0 - 1.5	T4 1.5 - 3.0	T5 3 - 6	T6 6 - 10	T7 10 - 17	T8 17 - 30	T9 30 - 50	T10 50 - 80
Chart B1	S1	2.94	3.15	3.35	3.59				
	S2	2.52	2.73	2.94	3.17				
	S3	2.10	2.31	2.51	2.74				
	S4	1.79	2.00	2.20	2.43				
	S5	1.58	1.69	1.79	2.02				
	S6	1.04	1.17	1.30	1.43				
Chart C1	S1				4.47	4.92	5.47		
	S2				4.00	4.45	5.00		
	S3				3.47	3.92	4.47		
	S4				3.16	3.61	4.06		
	S5				2.79	3.27	3.75		
	S6				2.44	2.92	3.40		
Chart C2	S1				4.30	4.90	5.49		
	S2				3.83	4.43	5.02		
	S3				3.46	3.94	4.41		
	S4				3.07	3.54	4.02		
	S5				2.94	3.29	3.63		
	S6				2.68	3.02	3.37		
Chart D1	S1				4.55	4.89	5.23	5.46	5.82
	S2				4.23	4.47	4.70	5.04	5.40
	S3				3.87	4.10	4.44	4.68	5.03
	S4				3.39	3.63	3.97	4.21	4.56
	S5				3.19	3.42	3.66	3.90	4.25
	S6				2.78	3.01	3.25	3.48	3.84

Table H.7 Modified Structural Numbers for Designed Structures

Chart No	Subgrade	T1	T2	T3	T4	T5	T6	T7	T8
		<0.3	**0.3 - 0.7**	**1.0 - 1.5**	**1.5 - 3.0**	**3 - 6**	**6 - 10**	**10 - 17**	**17 - 30**
Chart A1	S1	1.93	2.11	2.44	2.65	2.93	3.18		
	S2	2.09	2.32	2.52	2.73	3.02	3.26		
	S3	2.38	2.51	2.63	2.86	3.14	3.38		
	S4	2.64	2.77	2.87	3.10	3.28	3.52		
	S5	2.88	2.99	3.11	3.24	3.46	3.70		
	S6	2.77	2.89	3.15	3.28	3.38	3.51		
Chart A2	S1	2.13	2.26	2.38	2.51	2.83	2.98	3.10	
	S2	2.19	2.31	2.44	2.57	2.88	3.03	3.29	
	S3	2.50	2.50	2.63	2.83	3.15	3.42	3.55	
	S4	2.56	2.68	2.81	2.94	3.25	3.40	3.66	
	S5	2.88	2.88	3.01	3.13	3.32	3.45	3.70	
	S6	2.77	2.77	2.89	3.02	3.24	3.46	3.72	
Chart A3	S1	2.03	2.13	2.26	2.38	2.58	2.68	2.81	
	S2	2.22	2.31	2.44	2.57	2.77	2.96	3.09	
	S3	2.53	2.53	2.66	2.76	2.96	3.16	3.29	
	S4	2.51	2.61	2.73	2.93	3.16	3.35	3.48	
	S5	2.76	2.76	2.89	3.08	3.31	3.41	3.54	
	S6	2.77	2.77	2.89	3.02	3.15	3.28	3.41	

Table H.7. (continued). Modified Structural Numbers for Designed Structures

Chart No	Subgrade	T3	T4	T5	T6	T7	T8	T9	T10
		1.0 - 1.5	1.5 - 3.0	3 - 6	6 - 10	10 - 17	17 - 30	30 - 50	50 - 80
Chart B1	S1	2.49	2.70	2.90	3.14				
	S2	2.57	2.78	2.99	3.22				
	S3	2.70	2.91	3.11	3.34				
	S4	2.84	3.05	3.25	3.48				
	S5	3.08	3.19	3.29	3.52				
	S6	2.94	3.07	3.20	3.33				
Chart C1	S1				4.02	4.47	5.02		
	S2				4.05	4.50	5.05		
	S3				4.07	4.52	5.07		
	S4				4.21	4.66	5.11		
	S5				4.29	4.77	5.25		
	S6				4.34	4.82	5.30		
Chart C2	S1				3.85	4.45	5.04		
	S2				3.88	4.48	5.07		
	S3				4.06	4.54	5.01		
	S4				4.12	4.59	5.07		
	S5				4.44	4.79	5.13		
	S6				4.58	4.92	5.27		
Chart D1	S1				4.10	4.44	4.78	5.01	5.37
	S2				4.28	4.52	4.75	5.09	5.45
	S3				4.47	4.70	5.04	5.28	5.63
	S4				4.44	4.68	5.02	5.26	5.61
	S5				4.69	4.92	5.16	5.40	5.75
	S6				4.68	4.91	5.15	5.38	5.74

H.5 The AASHTO pavement design method

In addition to the analytical approach described above, comparisons were also made with the designs deriving from the AASHTO method. The design equation requires five input parameters. These are shown in Table H.8. The output of the method is a Structural Number.

Table H.8 The AASHTO method

Factor	Value	Comments
Reliability used in design equation	98%	The reliability is very strongly dependant on the value selected for S_0 but most of the variability comes from the subgrade. By using the measured 5 or 10-percentile rather than the average values, the design is more accurate and the residual variability represented by S_0 is low.
Effective reliability	90% or 95% depending on subgrade percentile selected	
Standard deviation (S_0) of performance	0.15	
Decrease in 'serviceability'	2.2	
Subgrade modulus	See Table H.2	
Strength coefficients for pavement materials	See Table H.4 above	

The AASHTO design equation itself is not applicable to all of the structures in this manual but where applicable (i.e. the designs incorporating HMA surfacings), the Structural Number values obtained using the input parameter values in Tables H.2, H4 and H8 are in close agreement. Higher levels of reliability and higher levels of terminal serviceability would, however, require thicker structures but the structures described herein are considered appropriate and the reliability can be adjusted conveniently by simply choosing a different subgrade CBR percentile for design.

Appendix I: DYNAMIC CONE PENETROMETER

I.1 Introduction

The Dynamic Cone Penetrometer (DCP) is an instrument which can be used for the rapid measurement of the in situ strength of existing pavements constructed with unbound materials. Measurements can be made down to a depth of approximately 800mm or, when an extension rod is fitted, to a depth of 1200mm. Where pavement layers have different strengths, the boundaries between them can be identified and the thickness of each layer estimated.

DCP tests are particularly useful for identifying the cause of road deterioration when it is associated with one of the unbound pavement layers, e.g. shear failure of the roadbase or sub-base. A comparison between DCP test results from sub-sections that are just beginning to fail and those that are sound will quickly identify the pavement layer which is the cause of the problem.

It is usually convenient to convert the individual pavement layer thicknesses and strengths measured in the DCP test into a structural number as described in Appendix H.

If it is suspected that the road failures are related to the *overall* structural strength of the pavement, the Modified Structural Number of different sub-sections can be readily compared to identify the weakness.

I.2 DCP test procedure

The TRL DCP uses an 8kg hammer dropping through a height of 575mm and a 60° cone having a maximum diameter of 20mm.

The instrument is assembled as shown in Figure I.1. It is supplied with two spanners and a tommy bar to ensure that the screwed joints are kept tight at all times. To assist in this the following joints should be secured with a non-hardening thread locking compound prior to use:

i) Handle/hammer shaft
ii) Coupling/hammer shaft
iii) Standard shaft/cone

The instrument is usually split at the joint between the standard shaft and the coupling for carriage and storage and therefore it is not usual to use locking compound at this joint. However it is important that this joint is checked regularly during use to ensure that it does not become loose. Operating the DCP with any loose joints will significantly reduce the life of the instrument.

I.3 Operation

A safe working environment should be maintained at all times. Many organisations will have on-site safety procedures which should be followed. Where there are no local safety procedures those in TRL's Overseas Road Note 2 are recommended.

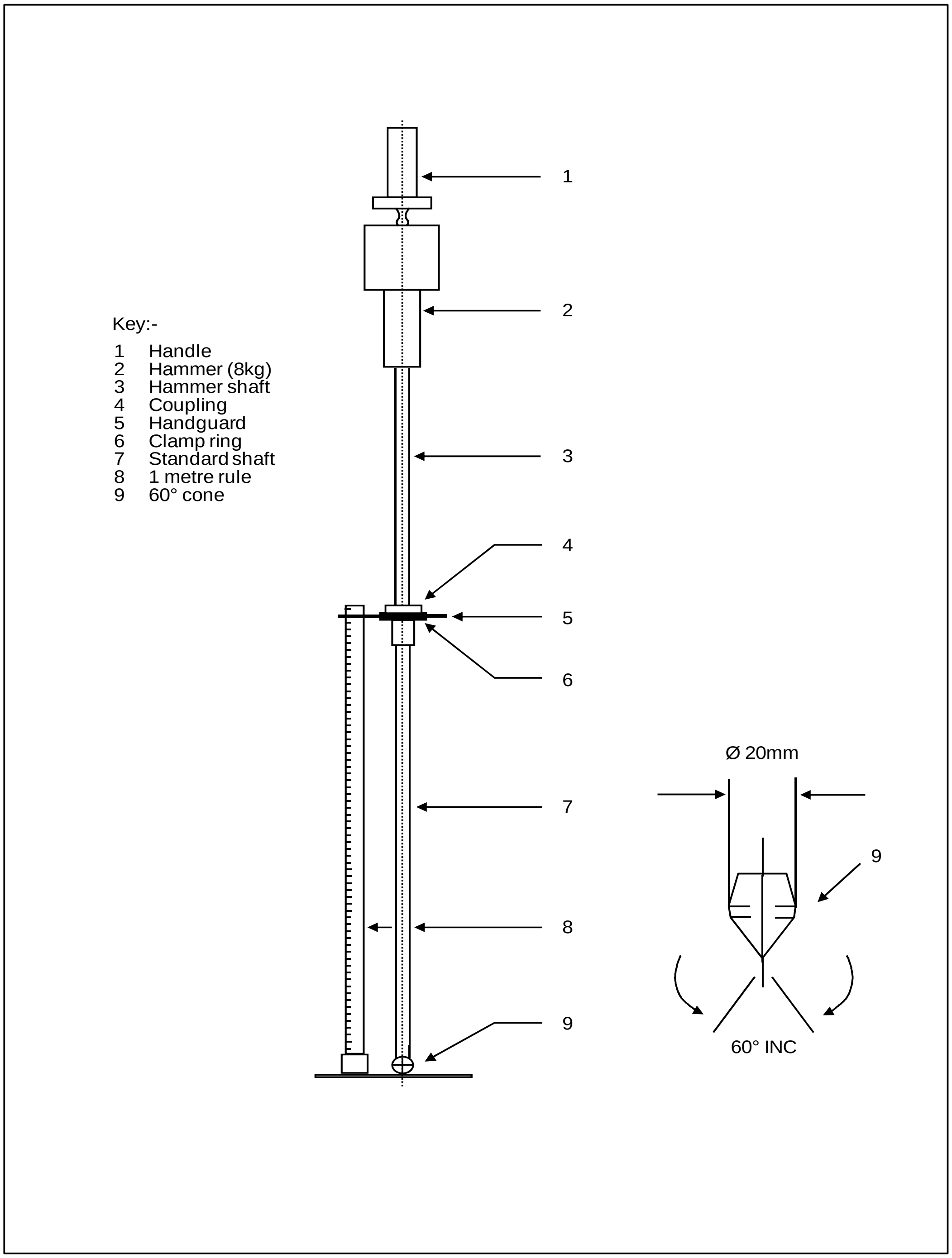


Figure I.1 Dynamic Cone Penetrometer

After assembly, the first task is to record the zero reading of the instrument. This is done by standing the DCP on a hard surface, such as concrete, checking that it is vertical and then entering the zero reading in the appropriate place on the proforma (See Figure I.2).

The DCP needs three operators, one to hold the instrument, one to raise and drop the weight and a technician to record the readings. The instrument is held vertical and the weight raised to the handle. Care should be taken to ensure that the weight is touching the handle, but not lifting the instrument, before it is allowed to drop. The operator must let it fall freely and not partially lower it with his hands.

It is recommended that a reading should be taken at increments of penetration of about 10mm. However it is usually easier to take a reading after a set number of blows. It is therefore necessary to change the number of blows between readings, according to the strength of the layer being penetrated. For good quality granular bases readings every 5 or 10 blows are usually satisfactory but for weaker sub-base layers and subgrades readings every 1 or 2 blows may be appropriate. There is no disadvantage in taking too many readings, but if readings are taken too infrequently, weak spots may be missed and it will be more difficult to identify layer boundaries accurately, hence important information will be lost.

When the extended version of the DCP is used the instrument is driven into the pavement to a depth of 400-500mm before the extension shaft can be added. To do this the metre rule is detached from its base plate and the shaft is split to accept the extension shaft. After re-assembly a penetration reading is taken before the test is continued.

After completing the test the DCP is removed by tapping the weight upwards against the handle. Care should be taken when doing this; if it is done too vigorously the life of the instrument will be reduced.

The DCP can be driven through surface dressings but it is recommended that thick bituminous surfacings are cored prior to testing the lower layers. This should be done using as little lubricating water as possible to avoid wetting the layer below and obtaining an incorrect strength reading. Little difficulty is normally experienced with the penetration of most types of granular or lightly stabilised materials. It is more difficult to penetrate strongly stabilised layers, granular materials with large particles and very dense, high quality crushed stone. The TRL instrument has been designed for strong materials and therefore the operator should persevere with the test. Penetration rates as low as 0.5mm/blow are acceptable but if there is no measurable penetration after 20 consecutive blows then it can be assumed that the DCP will not penetrate the material. Under these circumstances a hole can be drilled through the layer using an electric or pneumatic drill, or by *dry* coring. The lower pavement layers can then be tested in the normal way. If only occasional difficulties are experienced in penetrating granular materials, it is worthwhile repeating any failed tests a short distance away from the original test point.

If, during the test, the DCP leans away from the vertical, no attempt should be made to correct it because contact between the shaft and the sides of the hole can give rise to an overestimate of subgrade strength as a result of friction on the rod caused by either tilted penetration through, or collapse of, any upper granular pavement layers. Where there is a substantial thickness of granular material, and when estimates of the actual subgrade strength are required (rather than relative values) it is recommended that a hole is drilled through the granular layer prior to testing the lower layers.

It is always advisable to check that side friction has not influenced the result of a DCP test. This is easily done by attempting to twist the shaft when the DCP is at full penetration. If the shaft cannot be spun reasonably easily between the fingers then there is too much side friction and the test should be repeated. The erroneous test should be marked as such but not abandoned because the side friction problem may not have arisen for the first part of the test.

If the DCP is used extensively for hard materials, wear on the cone itself will be accelerated. The cone is a replaceable part and it is recommended by other authorities that it should be replaced when its diameter is reduced by 10 per cent. However, other causes of wear can also occur hence the cone should be inspected before every test.

DCP TEST

Site/Road:
Test No:
Section No/Chainage:
Direction:
Wheel path:

Date:

Zero reading of DCP:
Started test at:

No. Blows	Σ Blows	mm	No. Blows	Σ Blows	mm	No. Blows	Σ Blows	mm

Figure I.2 DCP Field Data Sheet

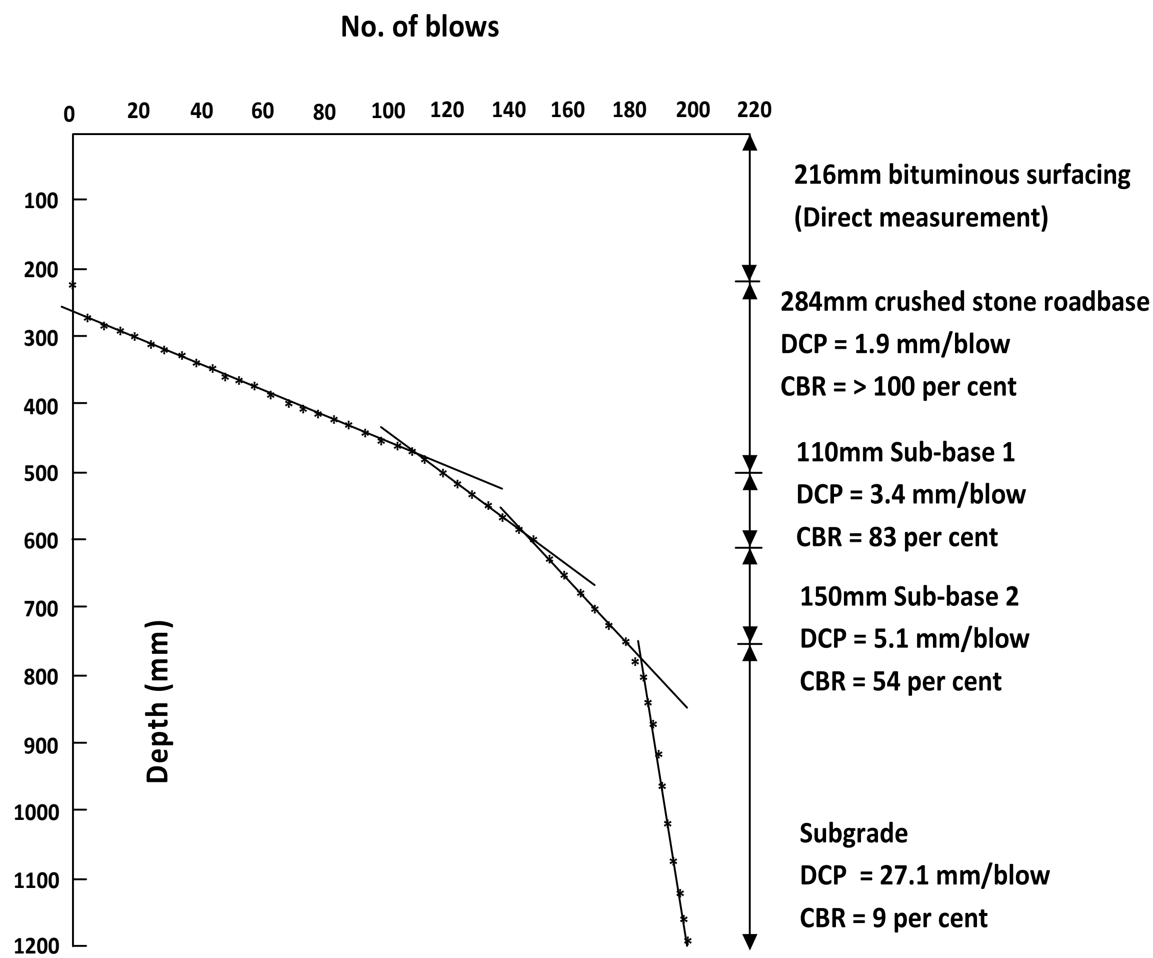
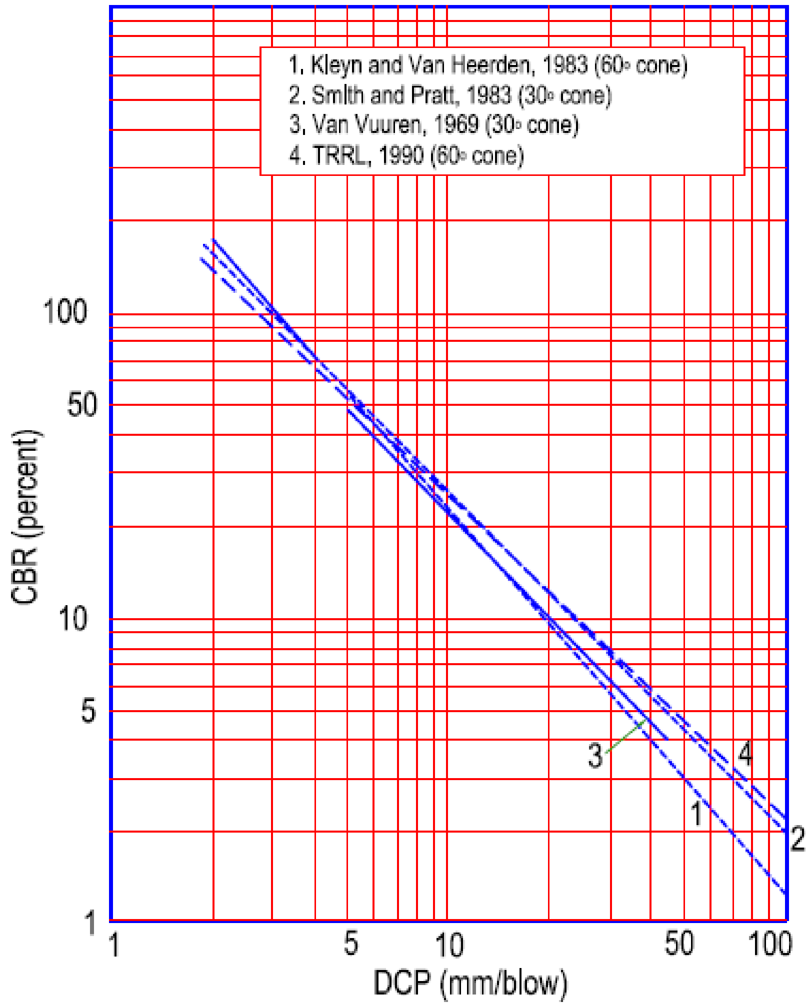


Figure I.3 Typical DCP test result

I.4 Interpretation of results

The results of the DCP test are usually recorded on a field data sheet similar to that shown in Figure I.2. The results can then either be plotted by hand, as shown in Figure I.3, or processed by computer.

Relationships between DCP readings and CBR have been obtained by several research authorities (see Figure I.4). Agreement is generally good over most of the range but differences are apparent at low values of CBR in fine grained materials. It is expected that for such materials the relationship between DCP and CBR will depend on material state and therefore, if more precise values are needed it is advisable to calibrate the DCP for the material being evaluated.



1. Log_{10} (CBR)=2,632-1,28Log_{10}(mm/blow)

2. Log_{10}(CBR)=2,555-1,145Log_{10}(mm/blow)

3. Log_{10} (CBR)=2.503-1.15Log_{10}(mm/blow)

4. Log_{10} (CBR)=2,48-1,057Log_{10}(mm/blow)

Figure I.4 DCP – CBR relationships

Appendix J: PROBE PENETRATION TEST

J.1 General Description

This test utilizes a modified soil assessment cone penetrometer, originally designed by the UK Military Engineering Experimental Establishment for the assessment of in-situ soil strength. The standard cone normally used with this penetrometer is replaced by a 4mm diameter probe rod with a hemispherical tip made of hardened steel. The probe is forced into the road surface under a load of 35kgf (343N) applied for 10 seconds and the depth of penetration is measured by a spring loaded collar that slides up the probe rod. The distance the collar has moved is measured with a modified dial gauge. The temperature of the road surface is recorded and a graphical method is used to correct the probe measurements to an equivalent value at a standard temperature of 30°C.

J.2 Method of Operation

All measurements are made in the nearside wheel track of each traffic lane where maximum embedment of chippings can be expected. A minimum of ten measurements is required at each location. These should be evenly spaced along the road at intervals of 0.5m, any recently repaired or patched areas being ignored. For convenience the measurement points can be marked with a chalk cross. The probe tip should not be centred on any large stones present in the road surface.

Before each measurement the collar is slid down the probe rod until it is flush with the end of the probe. The probe is then centred on the measurement mark and a pressure of 35kgf is applied for 10 seconds, care being taken to keep the probe vertical. The probe is then lifted clear and the distance the collar has slid up the probe is recorded in millimetres.

It sometimes occurs that the point selected for test is below the general level of the surrounding road surface. It is then necessary to deduct the measurement of the initial projection of the probe tip from the final figure.

The road surface temperature should be measured at the same time that the probe is used and the tests should not be made when the surface temperature exceeds 35°C. This will limit probe testing to the early morning in many locations. The probe readings are corrected to a standard temperature of 30°C using Figure J.1, and the mean of ten probe measurements is calculated and reported as the mean penetration at 30°C. Categories of road surface hardness and the corresponding ranges of surface penetration values are shown in Table 9.7.

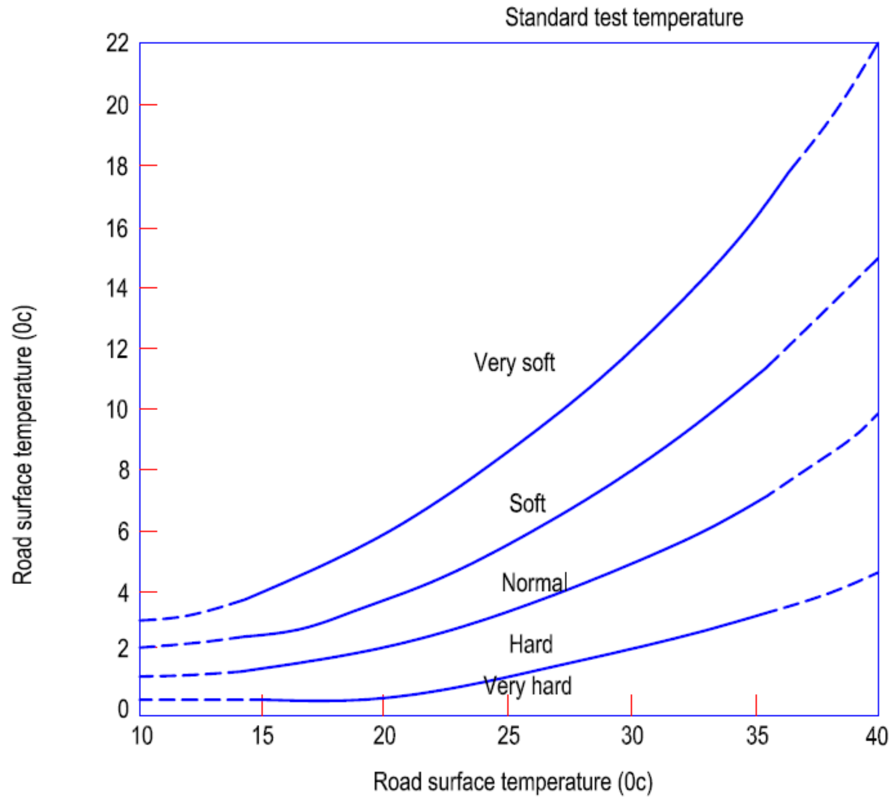


Figure J.1 Graphical Method for Correcting Measurements of Road Surface Hardness to the Standard Test Temperature of 30^{o}C

Appendix K: APPLICABLE STANDARDS FOR HMA

K.1 British Standards

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. The previous British Standards have therefore been referred to in these references. A list of provisional CEN standards is given below.

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. (British Standards Institution).

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.

K.2 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. A list of some of the standards is given in the following Tables.

Applicable CEN Standards

	Aggregates	**Old BS**
prEN 13043	Aggregates for bituminous mixtures and surface dressings for roads and other trafficked areas	
BS EN 932-1	Methods of sampling	BS 812-102
BS EN 932-2	Methods for reducing laboratory samples	BS 812-102
BS EN 932-5	Common equipment and calibration	BS 812-100
BS EN 932-6	Definitions of repeatability and reproducibility	BS 812-101
	Aggregates - Tests for geometric properties of aggregates	
BS EN 933-1	Determination of particle size distribution - Sieving method	BS 812-103
BS EN 933-2	Determination of particle size - Test sieves, nominal size of apertures	BS 812-103
BS EN 933-3	Determination of particle shape - Flakiness index	BS 812-105
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 crushing	BS 812-110 BS 812-111 BS 812-112
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 and abrasion value	BS 812-113 BS 812-114
BS EN 1367-2	Magnesium sulphate 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	
	Aggregates	
prEN 13043	Aggregates for bituminous mixtures and surface dressings for roads and other trafficked areas	
BS EN 932-1	Methods of sampling	BS 812-102
BS EN 932-2	Methods for reducing laboratory samples	BS 812-102
BS EN 932-5	Common equipment and calibration	
BS EN 932-6	Definitions of repeatability and reproducibility	
BS EN 933-1	Determination of particle size distribution - Sieving method	BS 812-103
BS EN 933-2	Determination of particle size - Test sieves, nominal size of apertures	BS 812-103
BS EN 933-3	Determination of particle shape - Flakiness index	BS 812-105
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	
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 812-2
BS EN 1097-7	Determination of the particle density of filler - Pyknometer method	BS 812-2
BS EN 1097-8	Determination of the polished stone value	
BS EN 1367-2	Magnesium sulphate test	BS 812-121
	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	
BS EN 1426	Determination of needle penetration	
BS EN 1427	Determination of softening point - Ring and Ball method	
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	BS598-102
prEN 12697-2	Test methods - Particle size distribution	BS598-102
BS EN 12697-3	Test methods - Bitumen recovery, rotary evaporator	BS2000-397
prEN 12697-5	Test methods - Maximum density	DD 228
prEN 12697-6	Test methods - Bulk density, measurement	BS598-104
prEN 12697-7	Test methods - Bulk density of bituminous specimens by gamma rays	
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 12697-13	Test methods – Compaction of rollers and bitumen temperature	BS598-109
prEN 12697-14	Test methods – Water content of bituminous mixtures	BS598-102
prEN 12697-15	Test methods - Segregation sensitivity	
prEN 12697-18	Test methods – Binder drainage	DD232
prEN 12697-19	Test methods - Permeability of porous asphalt specimen	
prEN 12697-22	Test methods - Wheel tracking	BS598-110
prEN 12697-23	Test methods - Indirect tensile test	
prEN 12697-24	Test methods – Resistance to fatigue	

prEN 12697-25	Test methods – Cyclic compression (for deformation resistance)	DD 213
prEN 12697-26	Test methods - Stiffness	DD 213
prEN 12697-27	Test methods - Sampling	BS 598 - 100
prEN 12697-28	Test methods - Preparation of samples for determining binder content, water content and grading	BS 598 - 101
prEN 12697-29	Test methods - Dimensions of a bituminous specimen	
prEN 12697-30	Test methods - Specimen preparation by impact compactor	BS598-107
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	BS598-107
prEN 12697-35	Test methods - Laboratory mixing	BS598-107
prEN 12697-36	Test methods – Thickness of bituminous pavement	
prEN 12697-37	Test methods – Properties of coated chippings	BS598-108
prEN 12697-38	Test methods - Common equipment and calibration	
prEN 12697-40	Test methods – Relative hydraulic conductivity of permeable surfacings	DD 229
prEN 12697-42	Test methods – Presence of foreign matter	
prEN 12697-43	Test methods - Resistance to fuel	
	Surface characteristics - Road and Airfield	
prEN 13036-1	Test methods - Measurement of pavement macro-texture depth using a patch technique	BS598-105
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	