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MINISTRY OF ROADS AND TRANSPORT

RDM 3.3

Road Design Manual

Volume 3: Materials and Pavement Design for New Roads

Part 3: Pavement Foundation and Materials Design

2025

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Foreword

This manual was developed by the Ministry pursuant to The Fourth Schedule of the Constitution which assigns to the National Government the functions and powers of setting standards for the construction and maintenance of all public roads including those under the County Governments.

It is part of a series of manuals that replace the first generation of road manuals developed in the first and second decades after independence. The second-generation road manuals were developed to cover the entire road project cycle covering planning, appraisal, design, contracts, construction, maintenance, operations and monitoring. The series incorporates best practices, climate change considerations, and recent technologies to enable safe, secure, and efficient road infrastructure.

Under the Kenya Vision 2030, infrastructure expansion and modernisation are one of the foundations for the realisation of Kenya's economic, social and political transformation into a rapidly industrialising middle-income country. The plan envisages an integrated, safe and efficient transport and communication infrastructure network consisting of roads, railways, ports, airports, waterways, and telecommunications infrastructure.

The strategies to be pursued under the Vision 2030 plan to improve infrastructure services and to maximise the economic and social impacts of infrastructure development and management include strengthening the institutional framework for infrastructure development and maintenance; raising efficiency and quality of infrastructure projects; Enhancing local content of identified infrastructure projects to minimise import content; Benchmarking infrastructure facilities and services provision with globally acceptable performance standards; and, Implementing infrastructure projects that will stimulate demand in hitherto marginalised areas.

The first three 5-year Medium Term Plan (MTP) under the Vision 2030 from 2008 to 2022 targeted construction of 1,950 km, 5,500 km and 10,000 km of new paved roads totalling 17,450 km. This was a massive infrastructure development program intended to double the paved road network in 10 years compared to the 8,600 km developed from independence in 1963 to 2008.

Implementing MTP I to III resulted in the construction of 14,000 km of paved roads, which was the phenomenal expansion of the paved road network that extended the paved road coverage to the Arid and Semi-Arid regions previously neglected. However, some key milestones of the Vision 2030 goals have not been realised. This has been due to internal and external challenges. External challenges included: climate change - prolonged droughts and floods; the emergence of the COVID-19 pandemic; global supply chain disruptions; exchange rate volatility; and rising interest rates in the leading economies.

The internal challenges included: inadequate road maintenance equipment; pavement overloading by heavy goods vehicles; huge maintenance backlog of the road network; low contracting and supervision capacity particularly in the Counties; poor quality control and assurance of works; congestion in urban areas; encroachment on road reserves; high cost and delays in payments of land acquisition; lack of harmonisation of cross-border transport regulation and operational procedures; rapid urbanisation; increased traffic volume with the exponential growth of motorcycle traffic; high cost/delays in relocation of utilities and services along and across road reserves; inadequate funding of projects and programs; and, delay or default in payments for goods, services and works.

The inability to address some of the above challenges is largely due to intrinsic systemic challenges which include: inadequate funding of research on roads and road construction materials; poor planning; lack of internalization of policies and processes; lack of respect for professionals and standard practice; ineffective coordination in the implementation of programs and projects; lack of inclusivity in engagement of manpower and procurement of services and works; and, lack of unity of purpose and synergy in development and delivery of projects.

The infrastructure expansion from 2008 – 2022 did not build the local contracting capacity (Micro, Small and Medium Enterprises) rather it destroyed them due to delays or defaults in payments of invoices at both national and county levels.

The implementation of MTP III came to an end on 30th June 2023, ushering in the implementation of the Fourth Medium Term Plan (MTP IV), which has been aligned with the aspirations of the Kenya Vision 2030 and the Bottom-Up Economic Transformation Agenda (BETA) planning approach and its key priorities.

BETA is the Government's transformation agenda geared towards economic turnaround through a value chain approach. BETA has targeted sectors with the highest impact to drive economic recovery and growth. This will be achieved by bringing down the cost of living; eradicating hunger; creating jobs; expanding the tax base; improving foreign exchange balances; and inclusive growth. BETA ensures rational resource allocation by eliminating wastage of resources occasioned by duplication, overlaps, fragmentation and ineffective coordination in the implementation of programmes and projects.

The Fourth Medium Term Plan key priorities are clustered under five key sectors, namely: Finance and Production; Infrastructure; Social; Environment and Natural Resources; and Governance and Public Administration. The infrastructure sector seeks to: enhance transport connectivity by constructing 6,000km of new roads, maintaining rural and urban roads, rail, air and seaport facilities and services; expanding communication and broadcasting systems; and promoting the development of energy generation and distribution by increasing investments in green energy (geothermal, wind, solar and hydro). The infrastructure gap is expected to be bridged by promoting economic participation of the private sector through public-private partnerships in the financing, construction, development, operation and maintenance of infrastructure.

BETA entails a shift of focus to fundamentals in project planning and implementation which include: respect for technical input, regulations and standard practices; adherence to project life cycle i.e., planning, feasibility studies and design before procurement of works; public and stakeholder consultation; procurement within budgetary ceilings; shifting focus during project implementation from the finished product ‘black top’ to the construction of the foundation; building local capacity particularly MSMEs by ensuring prompt payments; and capacity building at all levels to enable internalization of policies and processes.

The first generation of the road manuals were used for 35 to 45 years. It is my sincere hope that the second generation of the road standards which have been developed in alignment to the BETA approach will guide in solving most of the above challenges and those expected to emerge in the next 50 years. Implementation of the manuals will enable achievement of the BETA aspirations which include inclusive growth; creation of sustainable employment; building of MSMEs; climate change adaptation and realisation of the UN SDGs; enhanced efficiency in management of infrastructure and transport system; and, laying the foundation for the next national long-term plan at the end of the Vision 2030.

The second generation of the road manuals and specifications was prepared through an extensive consultative process involving review of existing standards and consultation with stakeholders, Ministerial Departments and Agencies, the Technical Task Force, public consultation and stakeholders’ workshops at review and drafting phases, and the National Steering Committee.

On behalf of the Government of Kenya, I would like to thank the African Development Bank for its support in the process of preparing this Manual. I would also like to thank the National Steering Committee, the Technical Task Force, the Technical Administrators, and the KeNHA Project Coordination Team for the sterling work done. I also thank the Consultant, TRL Limited for their role in providing technical expertise that was essential for the success of the Road manuals updating exercise.

Hon. Davis K. Chirchir, E.G.H
Cabinet Secretary, Ministry of Roads and Transport

Preface

This Part of Volume 3 covers the structural design of foundations and materials for both flexible pavement and rigid pavements for new roads in Kenya. Pavement foundations are designed on the basis of practical minimum layer thicknesses for construction, protection of the subgrade during construction and long-term provision of support to the overlying pavement layer. Other considerations include drainage and durability.

In the short term during pavement construction, the stresses in the foundation are relatively high. It is expected that loads are going to be applied to the foundation by delivery vehicles, pavers and other construction plant. At any level where such loading is applied, the stiffness and material thickness of the layer has have to be sufficient to withstand the load without damage occurring that might adversely influence, to any significant extent, the long-term performance.

In the longer term during the in-service life of a pavement, the stresses in the foundation are expected to be lower than during construction, although the foundation is going to experience repeated loads from traffic. It is essential that the assumed support of the foundation to the pavement is maintained, otherwise, deterioration of the upper pavement layers is going to occur more rapidly than anticipated.

This Part also covers the considerations and characteristic specifications of the materials suitable for use in pavement foundations and pavement layers in Kenya.

Before using this manual, the designer should have obtained subgrade data in accordance with RDM Volume 3 Part 1.

Eng. Joseph M. Mbugua, CBS
Principal Secretary, State Department for Roads

Document Management

Document Status

This document has the status of a Manual. Users shall apply the contents there-in to fully satisfy the requirements set out. The content of the manual is based on current practice in Kenya and the latest practices in the road sector, both regionally and internationally.

Sources of the Document

Copies of the document can be obtained from:
The Principal Secretary, State Department for Roads, Ministry of Roads and Transport, Works Building, Ngong Road, P.O. Box 30260 - 00100, NAIROBI Email: ps@road.go.ke

A secured PDF copy maybe downloaded from: www.roads.go.ke/downloads

Notification of errors and requests for amendments:

While all care and consideration has been applied in the compilation of this document, the Ministry accepts no responsibility for failure in any way related to the application of this manual or any reference documents cited in it.

Requests for edits and corrections can be freely sent to the following address:
The Principal Secretary, State Department for Roads, Ministry of Roads and Transport, Works Building, Ngong Road, P.O. Box 30260 - 00100, NAIROBI Email: ps@road.go.ke

Amendments Request Form Format

Request No.	Name	Organisation	Chapter	Page	Section/ Clause	Ref. to: Figure/ Table/	Type of Request	Request

Type of request: General – G; Editorial – E; Technical - T

Amendments to Date

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The technical work was undertaken under the guidance of a Technical Task Force, chaired by Eng. David Maganda, with the following gazetted members: Francis Gichaga (Prof.) (Eng.), Andrew Gitonga (Eng.), Timothy Nyomboi (Dr.) (Eng.), Rosemary Kungu (Eng.), Charles Obuon (Eng.), Sylvester Abuodha (Prof.) (Eng.), Samuel Kathindai (Eng.), Nicholas Musuni (Eng.), Charles Muriuki (Eng.), Tom Opiyo (Eng.), John Maina (Eng.), Fidelis Sakwa (Eng.), Daniel Cherono (Eng.), Maurice Ndeda (Eng.), Theo Uwamba (Eng.).

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Acronyms

AADT	Annual Average Daily Traffic
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
AC	Asphaltic Concrete
ACV	Aggregate Crushing Value
ADT	Average Daily Traffic
ALD	Average Least Dimension
AmSL	Above Mean Sea Level
B	Bus
BSM	Bitumen Stabilised Materials
BiSAR	Bitumen Stress Analysis in Roads
BS	British Standards
BSI	British Standards Institution
C	Car
CBR	California Bearing Ratio
CESA	Cumulative Equivalent Standard Axles
CIM	Cement Improved Material
CP	Concrete Pavement
CPB	Concrete Paving Block
CR	Crushing Ratio
CSIR	Council for Scientific and Industrial Research
CUSUM	Cumulatively Summed
CV	Commercial Vehicles
CV/d	Commercial Vehicles per day
DCP	Dynamic Cone Penetrometer
DESA	Daily Equivalent Standard Axles
DR	Dump Rock
DSD	Double Surface Dressing
EF	Equivalency Factor
EME-2	Enrobé à Module Élevé (EME2 - High modulus asphalt)
EML	Equilibrium Moisture Levels
ESA	Equivalent Standard Axles
ESM	Emulsion Stabilised Materials
ESP	Exchangeable Sodium Percentage
ETB	Emulsion Treated Base
FACT	Fines Aggregate Crushing Test
FI	Flakiness Index
GCS	Graded Crushed Stone

Acronyms

GDP	Gross Domestic Product
GM	Grading Modulus
GPS	Global Positioning System
H	Heavy Duty
HBM	Hydraulically Bound Material
HFS	Hubbard Field Stabilometer
HGV	Heavy Goods Vehicle
HIG	Hydraulically Improved Gravel
HPS	Hand Packed Stone
HRB	Hydraulic Road Binders
HSM	Hydraulically Stabilised Material
ICC	Initial Consumption of Cement
ICL	Initial Consumption of Lime
ICS	Interlocking Cobble Stone
ISO	International Standards Organisation
ITS	Indirect Tensile Strength
ITSdry	Indirect Tensile Strength conducted on a dry sample
ITSwet	Indirect Tensile Strength conducted on a soaked conditioned sample
KS	Kenyan Standard
L	Light Duty
LAA	Los Angeles Abrasion
LGV	Light Goods Vehicle
LL	Liquid Limit
LV	Low Volume
LVR	Low Volume Roads
LVSR	Low Volume Sealed Roads
M	Medium Duty
Mb	Minibus
MC	Medium Curing
Mc	Motorcycle
MDD	Maximum Dry Density
MESA	Million Equivalent Standard Axles
MGV	Medium Goods Vehicle
MLET	Multi-Layer Elastic Theory
MoRT	Ministry of Roads and Transport
MTRD	Materials Testing and Research Division
NMT	Non-Motorised Traffic
NPRA	Norwegian Public Roads Administration

Acronyms

OB	Omnibus
O-D	Origin – Destination
OMC	Optimum Moisture Content
PC	Pedal Cycle
PI	Plasticity Index
PL	Plastic Limit
PM	Plasticity Modulus (Product of PI and % passing 0.425 mm sieve).
RDM	Road Design Manual
SABitA	Southern African Bitumen Association
SADC	Southern African Development Community
SF	Seasonal Factors
SG	Specific Gravity
SMA	Stone Mastic Asphalt
SSD	Single Surface Dressing
SSS	Sodium Sulphate Soundness
TRL	Transport Research Laboratory
UC	Uniformity Coefficient
UCS	Unconfined Compressive Strength
USA	United States of America
VEF	Vehicle Equivalence Factor
VH	Vibrating Hammer
VPD	Vehicles Per Day
WBM	Water Bound Macadam

Abbreviations

cc	Cubic centimetres
h	Hour
kg	Kilogram
km	Kilometres
kN	Kilo Newton
kPa	Kilo Pascal
l	Litre
m	Metres
m^2	Meter Squared
m^3	Meter Cubed
mm	Millimetres
mm^2	Millimetres Squared
MN	Mega Newtons
MPa	Mega Pascal
N	Newton
Pen	Penetration
µm	Micrometre

Definitions and Glossary of Terms

General

Borrow Area/Pit	Is a site from which natural material, other than solid stone, is removed for construction of the works. The term borrow pit is applicable during and after the extraction of materials.
Cobblestone	Cobble or dressed stone surfacing consists of a layer of roughly rectangular to cubical dressed stone laid on a bed of sand or fine aggregate, within mortared stone or concrete edge restraints.
Concrete Pavement Blocks	Precast concrete blocks laid on a base of either flexible or rigid pavement. It can be laid in various patterns. They are manufactured in two categories of shapes, i.e., regular (R) or special (S) in accordance with Kenya standard KS 827.
Graded Crushed Stone	Is a base or sub-base material derived through crushing or rock or stone, conforming to the grading, strength, shape and soundness criteria given in materials specification Chart GM11 of this RDM.
Gravel Wearing Course	Is a top surfacing course made from gravel and applied to a road formation where no pavement or bituminous surfacing are to be placed. The term 'gravel' includes one or a combination of the following materials: lateritic gravel, quartzitic gravel, calcareous gravel, some forms of partly decomposed rock, soft stone, coral rag, clayey sands and crushed rock.
Hand Packed Stone	A process in which stones of maximum dimension ranging from 100 to 200 mm are packed by hand with the largest face downwards and the stones wedged with smaller stone; a fairly level surface on which layers of smaller graded stones are placed.
Hydraulically Bound Stone	Is a high-quality, well-graded aggregate, and cement mixture (or other hydraulic binder), mixed in a stationary plant and laid by a paver. It is used as a high-quality base.
Hydraulically Improved Granular Materials	Are natural sands and gravels, crushed stone gravel/aggregates or natural materials blended with crushed stone aggregates which are deficient in desirable properties, and which may be improved by the addition of either lime, cement or hydraulic road binder. Engineering properties such as strength and plasticity are improved but the material still remains flexible. Improved materials may be suitable for either sub-base or base.
Hydraulically Modified Stone	These are granular materials whose properties are enhanced by the use of hydraulic binders such as cement. The enhancement involves some limited binding of the large particles. It excludes material modified using bituminous binders.
Hydraulic Road Binder	Is a factory-produced hydraulic binder, supplied ready for use, having properties specifically suitable for the treatment of materials for bases, sub-bases and improved subgrade as well as earthwork materials (fill, selected material etc)
Hydraulically Stabilised Materials	Naturally occurring gravel or crushed rock whose strength is enhanced by the addition of either lime, cement or hydraulic road binders.
Quarry	Is an open surface working from which stone is removed by drilling and blasting, for construction of the works.
Rockfill	Is a rock material of such particle size that the material can only be placed in layers of compacted thickness exceeding 300 mm. Boulders with volumes greater than 0.2 m^3 are not normally used.

Bitumen and Bituminous Materials

Asphalt Concrete Surfacing	Is a group of bitumen-bound materials used as pavement surfacings. They normally consist of a mixture of coarse aggregate, fine aggregate and filler bound with a bituminous binder.
Asphalt Concrete Wearing Course	Is the upper layer when two-course asphalt concrete is used as a surfacing. It usually differs from the lower binder course, in having a slightly higher bitumen content and lower voids.
Asphalt Concrete Binder Course	Is the lower layer when two-course asphalt concrete is used as a surfacing. It usually differs from the upper, wearing course, in having a slightly lower bitumen content, lower stability and greater voids.
Bituminous Binders	Are petroleum-derived adhesives used to stick chippings onto a road surface in surface dressings, to bind together a layer of surfacing or base material, or to bind together aggregates in bitumen bound materials. There are four principal types used in road work (straight-run, cut-back, short residue, and emulsion):
Bitumen Emulsion	Is a binder in which petroleum bitumen, in finely divided droplets, is dispersed in water by means of an emulsifying agent to form a stable mixture.
Bitumen Stabilised Materials	Are pavement materials that are treated with either bitumen emulsion or foamed bitumen. The materials commonly treated include granular materials, previously cement or lime treated pavement materials and reclaimed asphalt pavements.
Cut-back Bitumen	Is a bitumen whose viscosity has been reduced by the addition of a volatile diluent.
Dense Bitumen Macadam	Is a hot-laid, plant mixture of well-graded aggregate, filler and straight-run bitumen, used for base construction. The resulting mix must conform to stability and flow criteria.
Dense Emulsion Macadam	Is a cold-laid, plant mixture of well-graded aggregate, filler and bitumen emulsion, used for base construction. The specifications are very similar to dense bitumen macadam.
EME (Enrobé à Module Élevé): [High Modulus Asphalt]	A mix technology developed in France providing high-performance asphalt material for use in heavy-duty pavements. Offers high-level resistance to permanent deformation combined with a high stiffness level, well in excess of that of standard mixes used in base layers, especially at elevated temperatures.
Emulsion Slurry Seal	Is a surfacing material, used by itself in one or two layers, or on top of a single surface dressing. It consists of fine aggregate, mineral filler and bitumen emulsion.
Foamed Bitumen	Hot bitumen temporarily greatly expanded in volume by the introduction of steam or water. It can be used in stabilisation or spray seal enrichment applications.
Fog Spray	Is a light application of bitumen emulsion or cut-back, on top of a surface dressing. Its purpose is to improve the waterproofness of the surfacing and to assist in holding the chippings.
Gap-Graded Asphalt	Is a hot-laid, plant mixture of gap-graded aggregate, filler and straight-run bitumen, used for pavement surfacing.
Polymer-Modified Bitumen	A binder consisting of polymeric materials dispersed in bitumen with enhanced binder performance for particular applications.

Prime Coat Is an application of low-viscosity bituminous binder to an absorbent surface, usually the top of the base. Its purposes are to waterproof the surface being sprayed and bind it to the overlaying bituminous course. The use of prime emulsion is preferred.

Sand Asphalt Is a surfacing material consisting of a hot-mixed, hot-laid, plant mixture of natural sand and, in some cases, mineral filler and crushed fine aggregate, bound with straight-run bitumen. It is not suitable for heavily trafficked roads.

Sand Bitumen Is a base material consisting of a cold, mixed-in-place combination of sand (or clayey sand) and either bitumen emulsion or cut-back. This material is intended for use in areas with little or no gravel deposits.

Short Residue Bitumen Is the primary product of the refinery before the air-blowing process and is a bitumen of variable viscosity whose penetration can be measured, approximating to a slow-curing cut-back.

Stone Mastic Asphalt A gap-graded wearing course mix with a high proportion of coarse aggregate, which interlocks to form a skeletal structure to resist permanent deformation. It has a high binder content.

Straight-Run Bitumen Is a bitumen whose viscosity or composition has not been adjusted by blending with solvents or any other substance.

Surface Dressing Is a method of providing a running surface to a pavement and consists of applications of bituminous binder and single-sized stone chippings. The usual form of this method on a new road is a double surface dressing with the second layer of chips being half the nominal size of the first. Single and triple surface dressings are also used.

Tack Coat Is a light application of bituminous binder to a bituminous or concrete surface to provide a bond between the surface and the overlaying bituminous course.

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1 Introduction

1.1 General

This manual was prepared by the Ministry as part of a series of manuals that cover the entire project cycle. The series incorporate best practices, climate change considerations, and recent technologies to enable the provision of road infrastructure that is safe, secure, and efficient.

The Kenya road manual series is as follows:

Project Cycle Stage	Manual: Volume or Part/Chapter	Code
A. General	**Procedures and Standards Manual**	**PSM**
	1. General	
	2. Policies	
	3. Procedures Guidance	
	4. Codes of Practice	
	5. Guidelines	
	6. Product/Testing Standards	
B. Planning	**Network and Project Planning Manual**	**NPM**
	1. Road Classification	
	2. Route/Corridor Planning	
	3. Route/Corridor Planning	
	4. Highway Capacity	
	5. Project Planning	
C. Appraisal	**Project Appraisal Manual**	**PAM**
	1. Environmental Impact Assessment and Audit	
	2. Social Impact Assessment	
	3. Traffic Impact Assessment	
	4. Road Safety Audits	
	5. Project Appraisal	
	6. Feasibility Studies	
D. Design	**Road Design Manual**	**RDM**
	1. Geometric Design	
	2. Hydrology and Drainage Design	
	3. Materials and Pavement Design for New Roads	
	4. Bridges and Retaining Structures Design	
	5. Pavement Maintenance, Rehabilitation and Overlay Design	
	6. Traffic Control Facilities and Communication Systems Design	
	7. Road Lighting Design	
E. Contracts	**Works and Services Contracts Manual**	**WSCM**
	1. Forms of contracts	
	2. Standard Specification for Road and Bridge Construction	
	3. Bills of Quantities	
	4. Standard/Typical Drawings	
F. Construction	**Road Construction Manual**	**RCM**
	1. Construction Management	
	2. Project Management	
	3. Site Supervision	
	4. Quality Assurance	
	5. Quality Control	

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Project Cycle Stage	Manual: Volume or Part/Chapter	Code
G. Maintenance	**Road Asset Management Manual**	**RAAM**
	1. Maintenance Management	
	2. General Maintenance	
	3. Pavement Maintenance	
	4. Bridges & Structures Maintenance	
H. Operations	**Road Operation Manual**	**ROM**
	1. Traffic Management	
	2. Vehicle Load Control	
	3. Emergency Services	
	4. Tolling	
I. Monitoring & Evaluation	**Road Design Manual**	**MEM**
	1. Performance Monitoring Manual	
	2. Technical Audits	
	3. Poverty, Gender Equality & Social Inclusion Monitoring	

This Volume 3, Part 3 – Pavement Foundation Design is part of the Roads Design Manual made up of a series of volumes and shown below:

Table 1.1 Road Design Manual (RDM) Coding Structure

Vol.	Manual Title	Part Name	Code
1	**Road Design Manual: Vol. 1** Geometric Design	**Part 1** - Topographic Survey	RDM 1.1
		Part 2 – Traffic Surveys	RDM 1.2
		Part 3 – Geometric Design of Highways, Rural and Urban Roads	RDM 1.3
2	**Road Design Manual: Vol. 2** Hydrology & Drainage Design	**Part 1** – Hydrological Surveys	RDM 2.1
		Part 2 – Drainage Design	RDM 2.2
3	**Road Design Manual: Vol. 3** Materials & Pavement Design for New Roads	**Part 1** – Ground Investigations and Material Prospecting	RDM 3.1
		Part 2 – Materials Field and Laboratory Testing	RDM 3.2
		Part 3 – Pavement Foundation and Materials Design	RDM 3.3
		Part 4 – Flexible Pavement Design	RDM 3.4
		Part 5 – Rigid Pavement Design	RDM 3.5
4	**Road Design Manual: Vol. 4** Bridges & Retaining Structures Design	**Part 1** – Geotechnical Investigation and Design	RDM 4.1
		Part 2 – Bridge & Culvert Design	RDM 4.2
		Part 3 – Retaining Structures Design	RDM 4.3
		Part 4 – Reinforced Fill Structures	RDM 4.4
		Part 5 – Bridges & Structures Condition Survey	RDM 4.5
		Part 6 – Bridge Maintenance Design	RDM 4.6
5	**Road Design Manual: Vol. 5** Pavement Maintenance, Rehabilitation & Overlay Design	**Part 1** – Pavement Condition Survey	RDM 5.1
		Part 2 – Pavement Maintenance, Rehabilitation and Overlay Design	RDM 5.2
6	**Road Design Manual: Vol. 6** Traffic Control Facilities & Communication Systems Design	**Part 1** – Road Marking	RDM 6.1
		Part 2 – Traffic Signs	RDM 6.2
		Part 3 – Traffic Signals and Communication System	RDM 6.3
		Part 4 – Other Traffic Control Devices	RDM 6.4
7	**Road Design Manual: Vol. 7** Road Lighting Design	**Part 1** – Grid-connected Road Lighting	RDM 7.1
		Part 2 – Solar Road Lighting	RDM 7.2

This volume must be applied sensibly and flexibly in conjunction with the skill and judgement of the designer. Compliance with the guidance given in the manual does not relieve designers of the responsibility for establishing that their design is suitable, appropriate, safe, and adequate for the purpose stated in the project requirements.

1.2 Objective of this Part

The main objective of this Part of Volume 3 is to provide options of materials to enable technically suitable and economically justifiable foundation design, and the structural design of both flexible and rigid pavements. The options selected must be suitable for the local conditions considering the availability of materials, technology, employment creation, health and safety, and efficiency of construction.

The options provided in this part may be adjusted by the designer followed by seeking the necessary approvals from the Chief Engineer (Materials).

Other objectives include traffic assessment for pavement design and materials design.

1.3 Scope of this Part

This Part sets out the permitted approaches for traffic assessment and materials characterisation for foundation and pavement design. A variety of materials can be utilised in the foundation and the designer can take advantage of improved foundation materials by using them to construct stronger and stiffer foundations to enable compliance with a modulus ratio requirement of at least 10 % to the overlying pavement construction and reduced pavement thickness.

Two foundation design approaches are presented:

1. A restricted foundation designs involves the use of subgrade surface moduli and characterisation of the improved subgrade or capping based on assumed performance provided in this manual and does not require verification via performance testing of the foundation.
2. A performance design approach that gives flexibility to the designer in terms of the materials that can be used in the foundation in conjunction with top of foundation testing to confirm that performance requirements have been met.

The foundation design covered under this part are for both flexible and rigid designs to be undertaken using Part 4 and Part 5 of this Volume.

1.4 Organisation of this Part

The remaining sections of the manual are structured as follows:

- Chapter 2: Traffic assessment. This discusses the steps required for the determination of the design traffic.
- Chapter 3: The Kenyan environment. This presents the climate, and geology of Kenya to provide a basis for providing appropriate design. Considerations for climate resilience are also presented here.
- Chapter 4: Classification of alignment soils. This provides the steps for the determination of uniform sections and the corresponding subgrade classes.
- Chapter 5: Earthworks. This provides guidance and considerations applicable to cuttings and embankments.
- Chapter 6: Problematic soils. This addresses methods for addressing the challenges presented by problematic soils found in Kenya.
- Chapter 7: Pavement foundations design. This provides cappings (improved subgrade) combinations required to achieve design foundation strength. Performance design of foundations is also discussed in this section.
- Chapter 8: Pavement Materials. This presents key aspects of various materials used in the design of flexible and rigid pavements and their specifications.

Appendices that contain sample axle loads and equivalence factors, the refusal density test procedure, the Bailey method of aggregate blending, a summary of the Marshall Mix design method, the modified Marshall method, and the Superpave mix design method are included.

1.5 Design Process

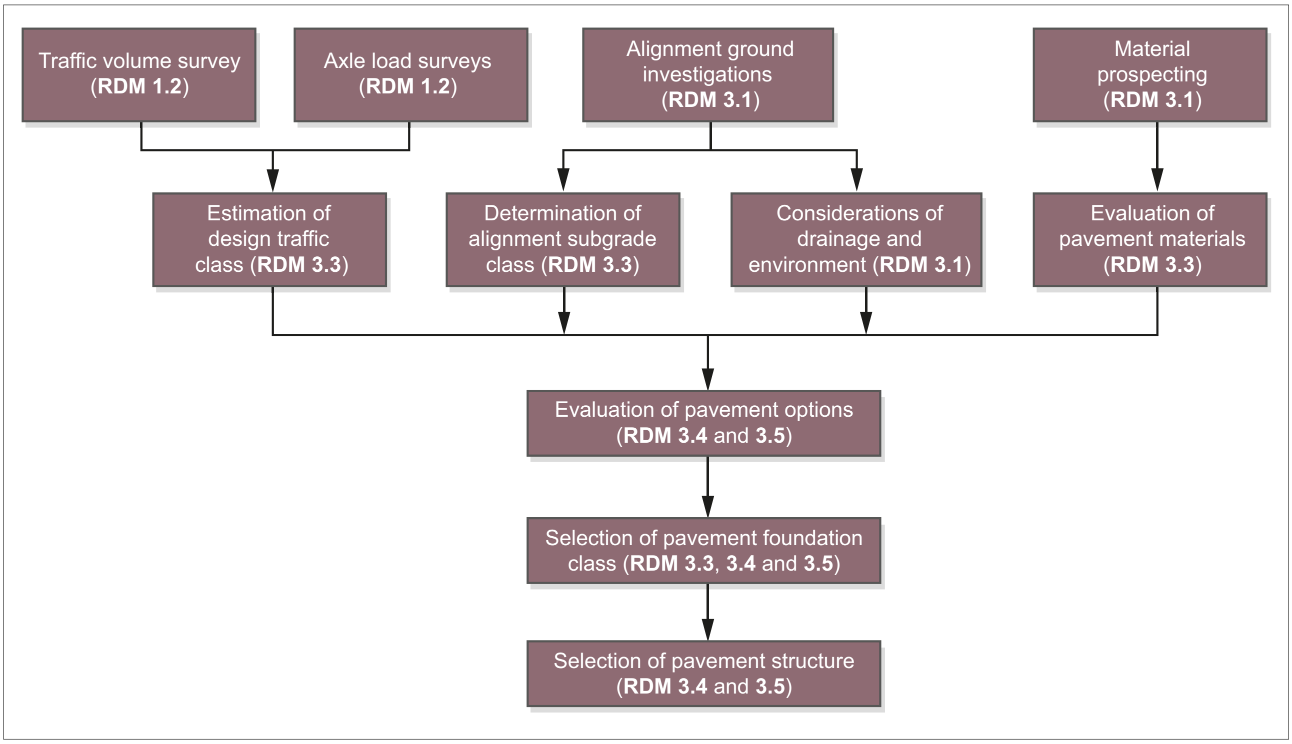
The main function of a pavement foundation is to distribute the applied vehicle loads to the underlying subgrade, without causing distress in the foundation layers or in the overlying layers. This is required both during construction and during the service life of the pavement.

Before using this manual, the designer should have obtained subgrade data in accordance with RDM Volume 3, Part 1.

Pavement foundations are designed on the basis of practical minimum layer thicknesses for construction, protection of the subgrade during construction and long-term provision of support to the overlying pavement layer. Other considerations include drainage and durability.

The design of the pavement structures included in this part is as presented in Figure 1.1 below.

Figure 1.1 The Design Process



1.6 Key Definitions

Figure 1.2 to Figure 1.4 illustrate the terms used in describing the principal pavement and cross section components.

Figure 1.2 Cross-section Terminology

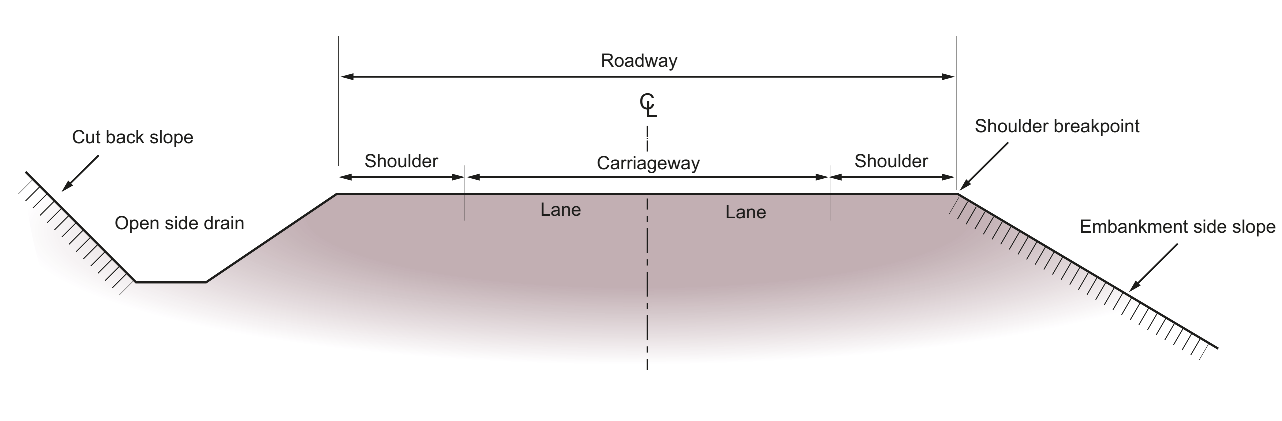


Figure 1.3 Cross-section Elements

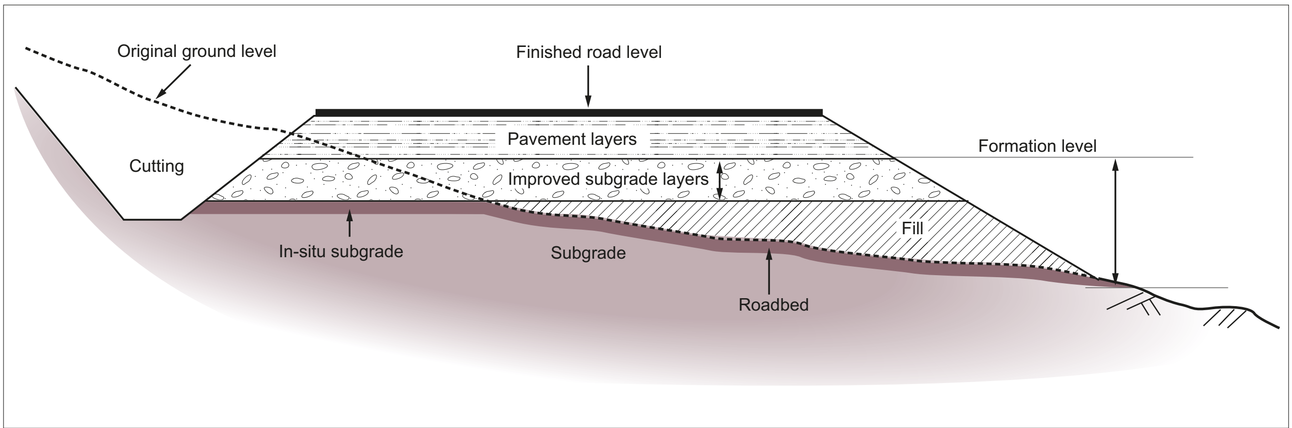
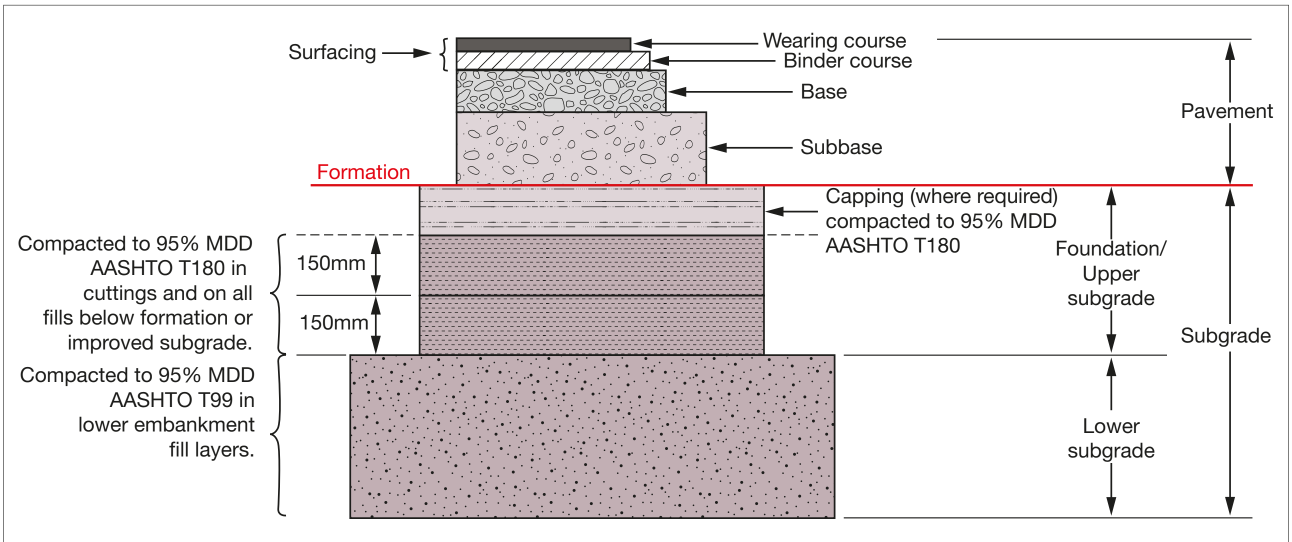


Figure 1.4 Pavement Terminology



Subgrade is all the material below the pavement and may include in-situ material, fill and capping (improved subgrade).

For assessing any section of subgrade, the average should be at least equal to the median for the category selected, and no CBR value should fall below the lowest value in the range.

Improved Subgrade or Capping is a layer of selected fill material, the top of which is at formation level, placed where the natural in-situ or fill material is unsuitable for the direct support of the pavement.

Formation is the surface of the ground, in its final shape, upon which the pavement structure, consisting of sub-base, base and surfacing is constructed.

Foundation is the upper subgrade below the formation. Where no capping (improved subgrade) is provided, it is the upper 300 mm of the subgrade below the formation. Where a capping layer is provided, the foundation shall be defined as the combination of the top 300 mm of fill or cutting and all the capping layers provided.

Sub-base is the layer constructed on the subgrade below the base either for the purpose of providing support to the base, and additionally protecting the subgrade. It may be composed of natural gravel, graded crushed stone, macadam, hand-packed stone, hydraulically improved granular materials, hydraulically modified stone, hydraulically bound stone, and bitumen stabilised material.

Base is a layer of material usually constituting the uppermost structural element of a pavement and on which the surfacing may be placed. It may be composed of natural gravel, graded crushed stone, macadam, hand-packed stone, hydraulically improved granular materials, hydraulically modified stone, hydraulically bound stone, bitumen stabilised material, or various forms of asphalt.

Surfacing is the uppermost pavement layer which provides the riding surface for vehicles. It will normally consist of one of the following: surface dressing, sand asphalt or asphalt concrete, stone mastic asphalt, other combinations of asphalt, cobblestone, and paving blocks. For rigid pavements, the concrete acts as both a base and surfacing.

2 Traffic Assessment for Pavement Design

2.1 General

A major factor in pavement design is the cumulative number of equivalent standard axles in the design period. In order to determine this value a number of operations must be carried out. This includes:

1. Conducting traffic counts and vehicle axle load surveys (covered in RDM Volume 1 Part 2).
2. Selecting a design period (in years).
3. Converting the axle loads to an equivalent number of standard 80 kN axles (covered in RDM Volume 1 Part 2).
4. Determining the daily number of standard axles for in the year of opening or initial daily number of standard axles.
5. Determining the cumulative number of equivalent standard axles over the design period and thus the determine the design traffic class.

2.2 Vehicle Classification

The vehicle classification system adopted for the traffic studies is shown in Table 2.1.

Table 2.1 Vehicle Classification System

Vehicle Category	Vehicle Class	Code	**Vehicle Description** based on unique characteristics and Traffic Act (Cap 403)	Class by Axle Configuration
Passenger Vehicles	**Pedal Cycle**	PC	Non-motorised bicycle or tricycle.	
	Motorcycle	MC	Self-propelled vehicle with less than 3 wheels.	
	Three wheelers and Tuk-tuk	MR	Self-propelled vehicle with 3 wheels.	
	Cars, jeeps, SUVs, Pick-ups	C	Passenger motor vehicle with seating capacity of not more than nine persons including the driver.	2-axle rigid
	Microbus	MCB	Two axle rigid chassis passenger motor vehicle with seating capacity of 10 to 14 persons including the driver.	2-axle rigid
	Minibus	MB	Two axle rigid chassis passenger motor vehicle with seating capacity of 15 to 25 persons including the driver.	2-axle rigid
	Bus	B	Two axle rigid chassis passenger motor vehicle with seating capacity of 26 to 53 persons including the driver.	2-axle rigid
	Omnibus	OB	Three or four axle passenger motor vehicles with seating capacity of more than 53 persons including driver.	3 or 4-axle rigid or articulated
Goods Vehicles	**Light Goods Vehicle**	LGV	Two axle rigid chassis goods vehicle of gross vehicle weight not exceeding 3,500 kg.	2-axle rigid
	Medium Goods Vehicle	MGV	Two axle rigid chassis goods vehicle or tractor of gross vehicle weight of 3,500 kg to 8,500 kg.	2-axle rigid
	Heavy Goods Vehicle	HGV	3 or 4 axle rigid chassis goods vehicle or tractor with gross vehicle weight greater than 8,500 kg.	3 or 4-axle rigid
	Articulated Heavy Goods Vehicle	AHGV	Articulated goods vehicle having 3 or more axles of gross vehicle weight exceeding 8,500 kg.	3 or more axles articulated

2.3 Permissible Axle Loads in Kenya

The prevailing allowable maximum axle loads prescribed under the Twelfth schedule at Rule 41 of the Traffic Rules under the Traffic Act (Chapter 403) of the Laws of Kenya are presented in Table 2.2. The recommended tyre pressure for design purposes is 750 kPa for dual tyres and 800 kPa for Super Single tyres.

Table 2.2 Permissible Maximum Axle Loads

Axle Type	Total Number of Wheels	Tyre Type	Maximum Axle Load (tonnes)
Single	2	Conventional	**8**
Single	4	Conventional	**10**
Tandem	8	Conventional	**18**
	4	Super Single	**16**
Tridem	12	Conventional	**24**
	6	Super Single	**22.5**
Liftable single	4	Conventional	**10**
Liftable single	2	Super Single	**8.5**

2.4 Selecting a Design Period

The concept of design period should not be confused with pavement life. At the end of the 'design period' the pavement will only require to be strengthened to carry traffic for a further period. At the end of the 'design period' the pavement will be **NOT** be completely worn out or have deteriorated to the point that reconstruction is needed.

During the design period of the pavement, only ordinary maintenance will be carried out, i.e., shoulders and drainage system maintenance, vegetation control, localised patching, and periodic resealing.

Pavement life refers to the length of time a pavement lasts to the point that strengthening is required to restore and extend serviceability. This is typically expected to occur after the design period but can occasionally occurs before the design period is reached.

The design aim is, therefore, to minimise the total expenditure on the pavement, including the initial construction costs and subsequent maintenance or strengthening costs discounted to present day value. This raises the question of stage construction.

For the types of pavements proposed in this Manual, stage construction offers economic advantages and initial design periods should not exceed the values in Table 2.3, even if much longer overall lives are anticipated. Stage construction provides an opportunity to choose the structural characteristics of the second stage in light of actual conditions, which may differ substantially from those originally foreseen.

The selection of design period depends on several factors and uncertainties. Based on the reliability of available information, the designer should specify the design period. Table 2.3 provides guidance on selection of design life. It should be noted that using Table 2.3 is an iterative process in which a design period is selected and used to compute the design traffic. The design traffic is then checked as to which design traffic category it belongs to. The DESA in Table 2.6 provides a good starting point for estimating the design traffic category. In each iteration, the design period is varied until the design period and the design traffic both correspond to the same design traffic category.

The values in Table 2.3 do not take into account designs for long-life (perpetual life) pavements. The design of these pavements is not based on design period, but rather on limiting strains at the bottom of bound layers (bituminous or hydraulic) and on top of the subgrade. For long-life pavements, the maximum permissible strain at the bottom of the bituminous layer is 70 microstrain, and at the top of the subgrade is 200 microstrain.

Bus-rapid-transit (BRT) lanes should be designed for a period of 40 years or until the pavement analysis shows that a long-life pavement has been achieved. Low and high levels of service refer to the perceived importance of the roads in question. This is determined by the relevant authority for the project road. For example, Class S, A, B, and C roads (see RDM 1.3) would require a high level of service whereas other roads would fall in low level of service.

Table 2.3 Pavement Design Period Selection Guidance

Design Traffic Category	Level of Service/ Design Period	
	Low	High
Low	10 – 15 years	15 years
Medium and Heavy Traffic	10 – 20 years	15 – 20 years
Very Heavy Traffic	20 years	30 years
Urban Pavements	15 - 20 years	40 years or long-life pavement
Rigid Pavements	40 years	40 years

2.5 Evaluation of Traffic for Design Purposes

To estimate the total number of standard axles to be catered for by the design, it is necessary to forecast the annual growth rate of the traffic and to decide what the design period should be as described below.

The calculation of design traffic loading shall include construction traffic and commercial vehicles that are expected to use the completed pavement before the start of the design period

The following steps are followed in determining the design traffic loading and class.

- **Step 1:** Select Design Period
  As described in section 2.4.
- **Step 2:** Estimate Initial Traffic Volume per Vehicle Class
  This should be for each direction as described in RDM Volume 1 Part 2.
- **Step 3:** Estimate Traffic Growth Rate per Vehicle Class
  A sensitivity analysis of the growth rates obtained in accordance with RDM Volume 1 Part 2 should be undertaken and an allowance of 1.2 times the growth rate should be made to cater for uncertainty if new developments and diverted traffic are expected along the route.
- **Step 4:** Mean ESA per Vehicle Class
  This should be for each vehicle class in each direction, as described in RDM Volume 1 Part 2.

  The equivalency factor (EF) for each axle is normally computed and summed up to obtain the EF for each vehicle type in the survey. The equivalency factor represents the mean damaging power of a vehicle to the pavement, normally expressed as the number of standard 80 kN (or 8,160kg) axles that would cause the same amount of damage.

  The EF (ESAs/axle) is derived as follows:

$$EF = \left(\frac{P}{8160}\right)^n \qquad \text{Equation 2.1}$$

Where,

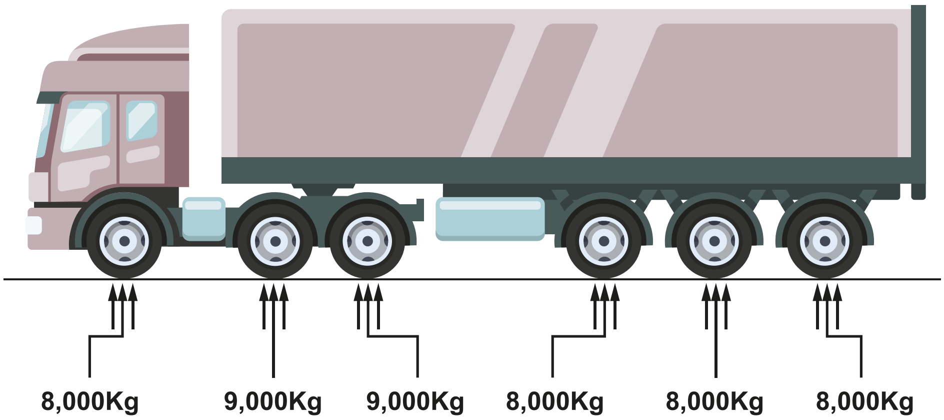
P = axle load (in kg)

n = power exponent

Example

Consider a 6-axle articulated truck with no tandem axles as shown in. If the truck is fully loaded, bringing the truck weight up to 50,000kg. The load per axle is as illustrated below.

Figure 2.1 A 6 Axle Semi-trailer (2:4:4:4:4:4)



The axle load measurements are converted into EF values using Equation 2.2. The VEF for one truck is:

$$VEF = \left(\frac{8000}{8160}\right)^{4.5} + 2 * \left(\frac{9000}{8160}\right)^{4.5} + 3 * \left(\frac{8000}{8160}\right)^{4.5} = 0.91 + 3.11 + 2.74 = 6.77$$

Equation 2.2

This means that one passage of the fully loaded semi-trailer exerts the same amount of pavement damage as 6.77 passages of a single axle with a load of 8,160kg.

The above relationship was derived by Liddle, taking a regional factor of 1.0 and a terminal serviceability index of 2.0. For Kenya, the default value of n shall be taken as 4.5. However, Liddle's formula does not hold for axle weights in excess of 130 kN. Empirical studies show that an equation is similar in form but with a damage exponent (higher than 4.5) depending on pavement structure and axle load may hold. The percentage of load carried by axles loaded to 13 tonnes and higher, expressed as proportion of the total mass of all vehicles surveyed should be computed. If this is equal or greater than 10 %, then a sensitivity analysis should be undertaken. This is conducted in the computation of the load equivalency factor by varying the value of the damaging exponent, n, from 3 to 9 for all axles measured. If these values lead to a design traffic class greater than the one obtained by a damaging exponent greater than 4.5, then the higher design traffic class should be adopted for the design.

All axles including tandem and triple ones should be weighed separately and the loads converted to equivalent standard axles using the above equation. The reasons are that there is some uncertainty concerning the tandem axle equation itself and that, in Kenya tandem axles appear to be improperly loaded (one axle is generally much heavier than the other) and so would not fulfil the equation requirements. The equivalence factors depend to some extent on the strength of the pavement.

It is important to note that the Liddle relationship is not linear but exponential. What this means is, small increases in the axle weight can result into large increases in pavement damage. For example:

1. A 44.4 kN single axle needs to be applied to a pavement structure more than 12 times to inflict the same damage caused by one repetition of an 80 kN single axle.
2. An 80 kN single axle does over 3,000 times more damage to a pavement than an 8.9 kN single axle (1.000/0.0003 ˜ 3,333).

   Individual wheels do slightly less damage when part of a tandem set than if they were widely separated. A 133.3 kN single axle does about 11 times more damage than a 133.3 kN tandem axle (7.9/0.703 ˜ 11). However, tandem, or triple axle groups rarely carry equal loads and therefore it is recommended to treat each axle of the tandem or triple set separately.

- **Step 5:** Mean Daily ESA for all Vehicle Classes

  The estimated initial daily ESAs for each vehicle class (DESA), for each direction of travel, is obtained from the traffic data derived in Step 2 and the VEFs derived in **Step 4**, as follows:

$$DESA_i = AADT_i \times VEF_i$$

Equation 2.3

Where,

i = vehicle class/category

$AADT_i$ = the AADT or ADT of vehicle class/category i, (**Step 2**)

VEF_i = the weighted average of VEF of vehicle class/category i, computed from axle load measurements as explained in **Step 4**. Typical examples of vehicle equivalency factors for the Kenya road network are provided in Appendix A.

- **Step 6:** Adjustment of the Computed DESA

After computation of the DESA in **Step 5**, the DESA should be adjusted for each vehicle class based on the recommendations in Table 2.4. The term heavy commercial vehicle shall refer to a commercial vehicle whose tare weight exceeds 3048 kg.

Table 2.4 Lane Width Adjustment Factors for Design Traffic Loading

Number of Carriageways	Carriageway Width	Traffic To Be Considered	Explanatory Notes
Single Carriageway	Less than 4.5m	The sum of DESAs in both directions for each vehicle class.	Traffic in both directions uses the same lane, but not all in the same wheel tracks as for a narrower road.
	Min. 4.5m but less than 7m	80% of the DESAs in both directions for each vehicle class.	To allow for overlap in the centre section of the road.
	7m or wider	Total DESAs in the heaviest loaded direction for each vehicle class.	Minimal traffic overlap in the centre section of the road.
	More than one lane in each direction	70% of the total DESAs in the studied direction for each vehicle class.	The majority of vehicles use one lane in each direction.
Dual Carriageways	Lanes of minimum 3.5 m width *(less than 2000 CV/d)*	The total commercial traffic in ONE DIRECTION	In such cases, the commercial vehicles are most likely to use the outermost lane.
	Lanes of minimum 3.5 m width *(more than 2000 CV/d)*	A special study of the distribution of traffic will be necessary	In such cases, the commercial vehicles are likely to utilise all available lanes in a bid to obtain better level of service.

1

2

Traffic Assessments for Pavement Design

- **Step 7:** Cumulative ESA (CESA) for all Vehicle Classes over the Design Period

  The cumulative equivalent standard axles (CESA) for each vehicle category expected over the design life of a road may be obtained from Equation 2.4.

$$CESA_i = 365 * \text{DESA}_i \left(\frac{(1 + r)^N - 1}{r}\right)$$

Equation 2.4

  Where,

  $CESA_i$ = Cumulative equivalent standard axles for vehicle class i.

  $DESA_i$ = Average daily ESAs for each vehicle class in the first design year (**Step 5**).

  r = Assumed annual growth rate expressed as a decimal (**Step 3**).

  N = Design period in years (from **Step 1**).

  This computation must consider different growth periods (which have different growth rates) for the entire design period. The CESA for each growth period is then summed to obtain the representative CESA for each vehicle class over the design period.

- **Step 8:** The Pavement Design Traffic

  The pavement design traffic is the sum of the cumulative equivalent standard axles for all vehicle classes considered for the pavement design of the particular road. It is computed as follows (Equation 2.5).

$$Pavement\ Design\ Traffic\ Loading = \sum_{i=1}^{m} CESA_i$$

Equation 2.5

  Where,

  $CESA_i$ = Cumulative equivalent standard axles for vehicle class i.

  i = Vehicle class/category.

  m = The total number of vehicle class/categories considered for design.

After the pavement design traffic is computed, the design traffic class is obtained from Table 2.6.

In Step 4 above, the standard axle load used is 80 kN (8160 kg), and this represents a single axle with dual wheels. For special projects as deemed by the Chief Engineer Materials, the determination of equivalency factor may be based on axle groups. In which case the 80 kN standard axle loads used in

Equation 2.1 will be replaced with the values shown in Table 2.5 depending on the axle groups. Axle load data from weighbridges comes in the format of axle groups weighed, hence it is easily applied in such a case.

Table 2.5 Standard Axles for Different Axle Load Groups

Tyre Type	Axle Group	Standard Axle Load (kg)
Dual Tyre Axles	Single Axle with Dual Tyres	8,160
	Tandem Axle with Dual Tyres	13,700
	Triaxle with Dual Tyres	18,500
	Quad Axle with Dual Tyres	23,000
Super-single (wide base) **Tyre Axles**	Single Axle with Single Tyres	5,900
	Tandem Axle with Single Tyres	9,990
	Triaxle with Single Tyres	13,400
	Quad Axle with Single Tyres	16,700

2.6 Design Traffic Classes

2.6.1 Use of Cumulative Number of Standard Axles

Attempts were made to define axle classes based on numbers of commercial vehicles. They have been largely unsuccessful because axle spectra vary considerably from one road to another. It has been found that the respective proportions of buses, medium goods and heavy goods vehicles on urban and suburban sections were completely different from those on rural roads.

A more rational approach is to base the traffic classification on the cumulative equivalent standard axle values. Even this method, based on the equation given in Chapter 2.4.3 has its drawbacks in that the initial axle load spectrum is assumed to remain fixed. Where it is fairly certain that some future event will alter the distribution of axle loads, this should be taken into account.

This type of classification enables the effects of unexpected changes in traffic volumes or axle-load distributions on the pavement life to be evaluated.

2.6.2 Traffic Classes

Traffic flow and axle-load surveys have shown that the following classes (Table 2.6) satisfactorily account for all traffic categories likely to be carried by the various classes of roads in Kenya.

Table 2.6 Lane Width Adjustment Factors for Design Traffic Loading

Design Traffic Class	Cumulative Equivalent Standard Axles	Design Traffic Loading (Million CESA)	Traffic Load Category	ESA/day (DESA): in Year one[2]	
				Min.	Max.
TC0.025	< 25,000	0.025	Low	0.0	0.2
TC0.10	25,000 - 100,000	0.10		0.2	8
TC0.25	100,000 – 250,000	0.25		8	21
TC0.50	250,000 – 500,000	0.50		21	41
TC1	500,000 – 1 million	1		41	83
TC3	1 million – 3 million	3	Medium	83	249
TC10	3 million – 10 million	10		249	829
TC17	10 million – 17 million	17	Heavy	829	1409
TC30	17 million – 30 million	30		1409	2486
TC50	30 million – 50 million	50		2486	4143
TC80	50 million – 80 million	80	Very Heavy	4143	6629
TC150	80 million – 150 million	150		6629	12428
TC150+[1]	> 150 million	Based on specific value		>12428	

Notes:

1. The "150+" in TC150+ class requires specific considerations and performance design based on the actual value of design traffic determined. In preparing their reports, the designer should then refer to the class based on the specific design traffic determined. For example, if the traffic determined is 160 MCESA, the design class will be TC160.

2. The DESA in the First year after opening to traffic (Year one) computed from the design traffic loading taking into account a constant growth rate of 5% over a design period of 20 years.

3 The Kenyan Environment

3.1 General

It is important that any designer is familiar with the environment in which their project site is located. This chapter discusses the climate and geology of Kenya at an introductory level. Additionally, an overview of climate change and climate resilience measures are given in this Chapter.

3.2 Climate

Climate has a considerable influence on road performance and should therefore be taken into account for by the design engineer during selection of pavement materials, construction methodology and maintenance interventions. These include:

1. Moisture variation in subgrade and pavement layer affecting bearing strength and pavement life which determine selection of sub grade and pavement materials.
2. Effects of temperature during construction and the performance of the various pavement materials incorporated in the road which informs the selection of pavement materials, construction methodology and maintenance interventions.
3. Problematic sub grade properties related to the environment.

Kenya lies between latitudes 5°N and 5°S, and longitudes 34° and 42°E, with a land area of approximately 580,367 km^2. Kenya has a very wide variety of climates, comprising:

1. Afro-alpine climate.
2. Equatorial climate.
3. Wet-tropical climate.
4. Semi-arid climate.
5. Arid climate.
6. Very arid climate.

Moreover, the pattern of the climatic zones is rather complex since the Kenyan climates are largely governed by altitude.

This diversity is illustrated by the Mean Annual Rainfall map and by the Air Temperature Charts given in Figure 3.1 and Figure 3.2. Detailed climatic data (rainfall, evaporation, and temperature) may be obtained from Kenya Meteorological Department.

The design of drainage and anti-erosion systems is largely dependent upon the climatic conditions. The choice of road making materials is equally influenced by climatic factors. In this respect the following guidelines should be followed by the design engineer:

1. In 'wet' areas (mean annual rainfall greater than 500 mm), the use of plastic pavement materials, should be avoided if possible. Bituminous surfacing should be as impervious as possible. Shoulders should be impermeable or properly sealed. Great attention should always be paid to both internal and external drainage. The subgrade strength shall be assessed at 4-days soaked CBR or until no further swell occurs.
2. In 'dry' areas (mean annual rainfall less than 500 mm), higher plasticity can be accepted for pavement materials and open-textured base materials can be used. However, the subgrade strength shall still be assessed at 4-days soaked CBR or until no further swell occurs. Difficulties may occur with cement-treated materials, because of the rapid evaporation of water hindering the hydration of cement and the tendency of the treated material to crack extensively as a result of shrinkage and volumetric changes caused by the daily temperature variations. Drainage and protection against erosion should not be neglected as short but heavy storms are likely to occur even in the driest areas.

Figure 3.1 Map Showing Temperature Zones

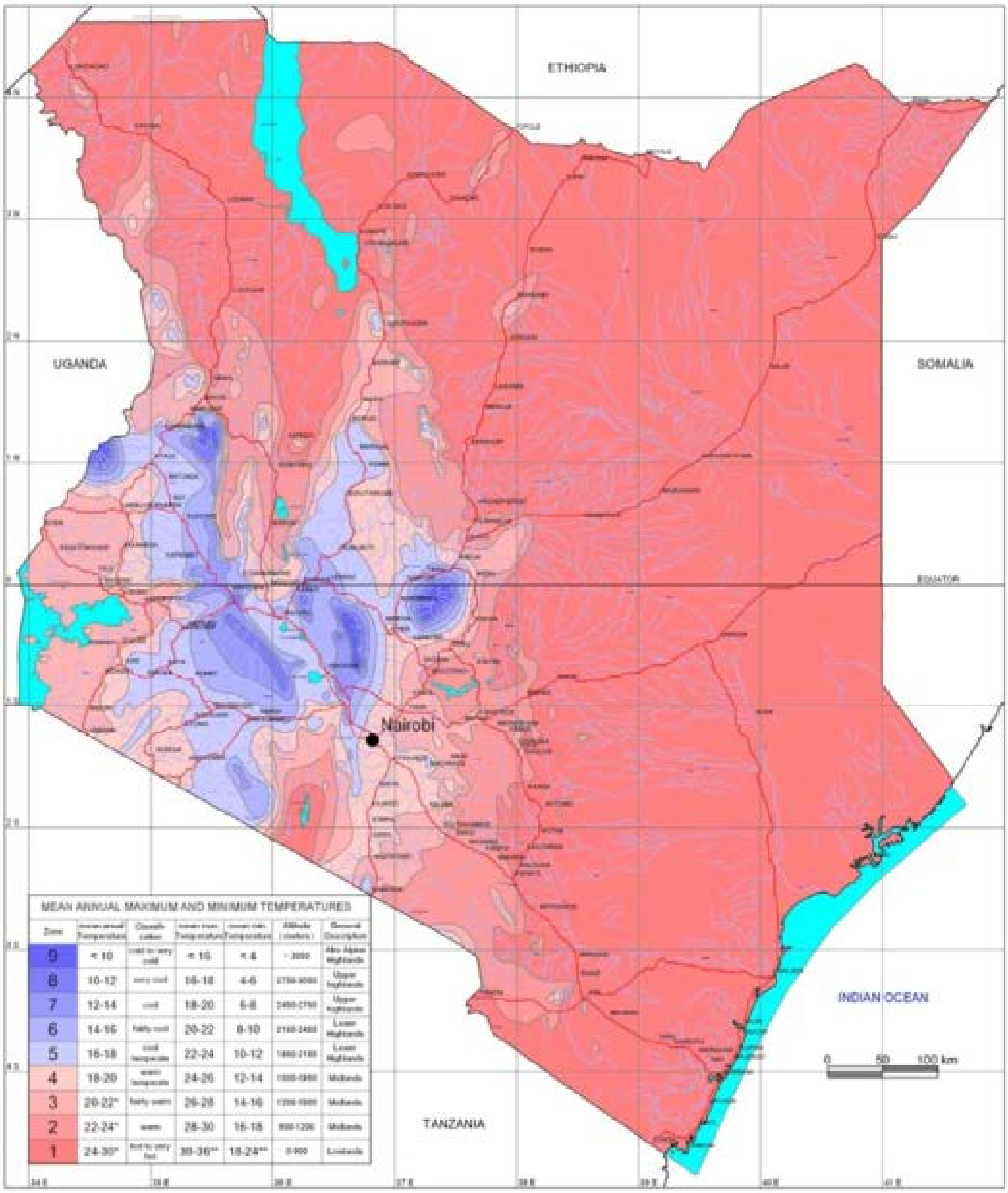
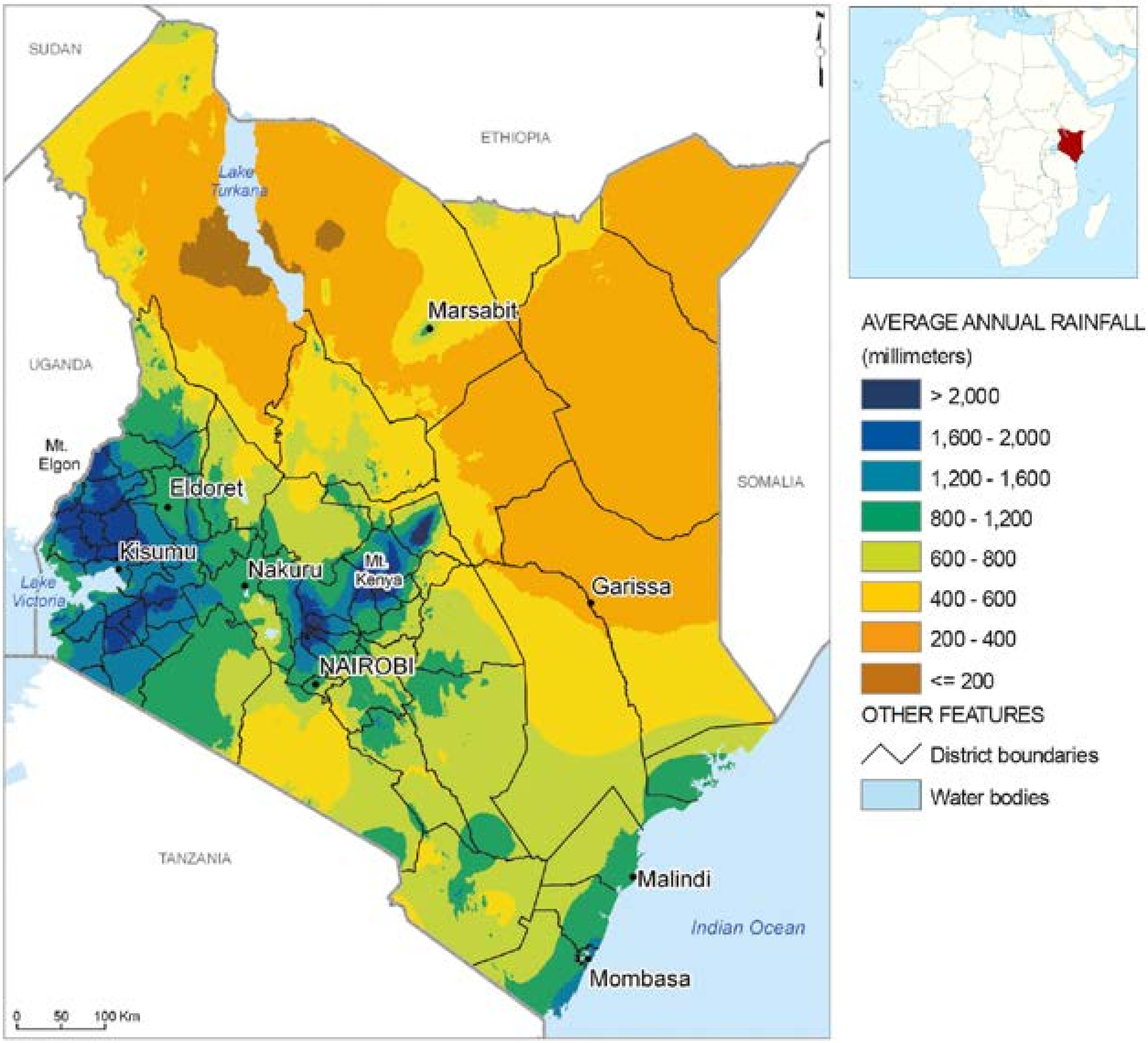


Figure 3.2 Map Showing Rainfall Zones



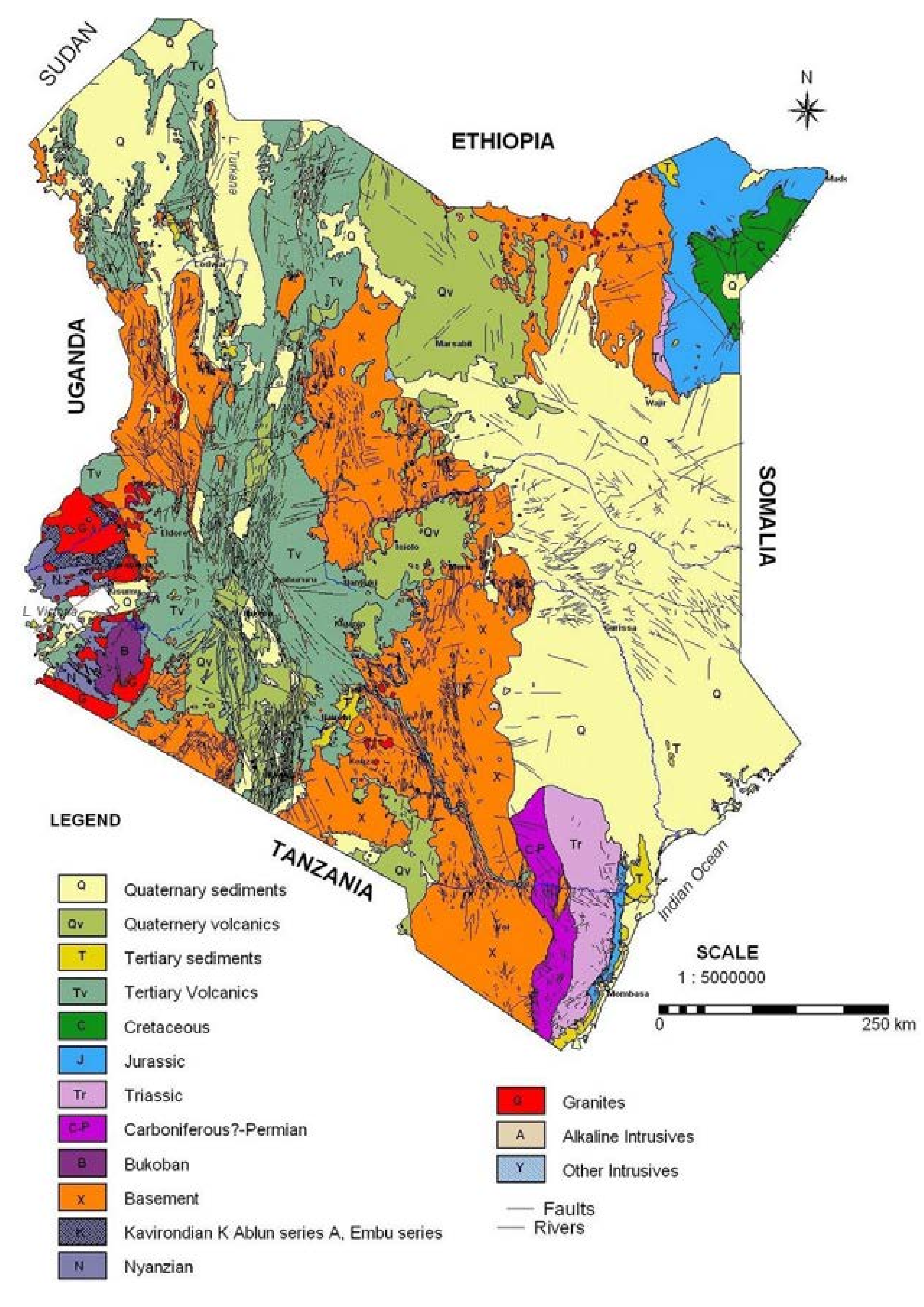
3.3 Geology

The geology of Kenya is characterised by Archean granite/greenstone terrain in western Kenya along Lake Victoria, the Neoproterozoic 'Pan-African' Mozambique Belt, which underlies the central part of the country and Mesozoic to recent sediments underlying the eastern coastal areas. The Eastern Rift Valley crosses Kenya from north to south and the volcanics associated with rift formation largely obliterate the generally north-south striking Neoproterozoic Mozambique Belt. Rift Valley volcanogenic sediments and lacustrine and alluvial sediments cover large parts of the Eastern Rift. A generalized geological map of Kenya is presented in Figure 3.3. A detailed geological map may be obtained from the Directorate of Geological Surveys of the Ministry of Petroleum and Mining, or from the National Geodata Centre (NGDC). An interactive map is available from their website.

A detailed soil map may be obtained from the Ministry of Agriculture and Livestock Development, or from the Regional Centre for Mapping of Resource for Development (RCMRD). An interactive map is available on their website.

From the coast on the Indian Ocean the low plains rise to central highlands, thence to the shores of Lake Victoria at 1130 mASL. The highlands are bisected by the Great Rift Valley but are also the site

Figure 3.3 Generalised Geological Map of Kenya



of the highest point in Kenya, Mount Kenya, which reaches 5,199 mASL. Mount Kilimanjaro which at 5,895 mASL is the highest point in Africa can be seen from Kenya just south of the Tanzanian border.

Kenya has abundant resources of hard stone. Detailed information regarding the various types of stone available and their road making characteristics can be found in Materials Branch Report No. 336.

Many different sorts of gravels exist in Kenya: lateritic gravels, quartzitic gravels, calcareous gravels, some forms of weathered rock, soft stone, coral rag, etc. Various types of sand and silty or clayey sands are also found. Detailed information concerning these materials and their engineering properties can be found in Materials Branch Reports No. 343 and 344.

The characteristics of the natural soils are discussed in Chapter 4, Sub grade and Earthworks, of this Manual.

In order to minimise construction costs, natural materials should be used as much as possible. Every endeavour should be made to use the cheap local materials before considering the importation of material from some distance. It is therefore of prime importance to make a complete inventory of all available road making materials, such as stone, gravel, sand, and clayey sand at the investigation stage.

3.4 Effects of Climate Change and Resilience Measures

The effect of climate change means changes have to be made in the way pavements are designed if the design life of the road is to be achieved. Due to increased cost of climate-proofing pavements, the interventions need to be applied after risk assessments of sections of road. Only sections at high risk should be proofed. Vulnerability assessment is discussed in RDM Volume 2 Part 1. If steps are not undertaken, this can lead to catastrophic failures of the pavement layers by effects of increased rainfall or temperature. Some of the steps taken in this manual include:

1. All subgrades are assessed at 4 days' soaked strength whether in wet or dry areas.
2. Subgrade class S3 (CBR 7-13 %) has been adopted as the minimum material for pavement foundation to avoid use of the lower subgrade materials which are more susceptible to moisture variation due to high plasticity and high swell properties.
3. Application of surface dressing seal on all asphalt concrete surfacing.
4. Plasticity specifications for granular materials and hydraulically modified materials have been made more stringent to minimise susceptibility to moisture variations. This may necessitate mechanical stabilisation of natural gravels which may not meet the specification using natural gravel of low plasticity or crushed stone aggregates.
5. Use of hydraulically modified granular materials that are less susceptible to moisture.

Climate mitigation measures such as planting trees in road reserves are encouraged.

In addition to the above, the designer should consider adopting the following approaches on vulnerable sections of any project road:

1. For low volume sealed roads, the surfacing should be made of triple surface dressing or single surface dressing on cold mix asphalt. This will minimise rainwater ingress. Hard grade base binders emulsions (50/70 penetration) should be used as substitutes for soft-grade binders to provide resilience against temperature increases.
2. Base and sub-base materials for low volume sealed roads should be graded crushed stone for resilience against ground water effects and arboriculture, provided they do not interfere with the road designed sight distances.

For medium to very heavy traffic, to provide resilience against high temperatures, a surfacing of double surface dressing on dense bitumen macadam (maximum stone size 25 mm) or stone mastic asphalt should be used. These alternatives also protect against rainwater ingress and are suitable for areas of severe axle loading. The standard bases and sub-bases provided (in Chapter 6) for these roads are generally resilient against ground water effects.

4 Classification of Alignment Soils

4.1 General

Before classifying subgrades based on strength, they should first be classified by soil type as described in RDM Volume 3 Part 1, and the road divided into homogeneous sections based on the soil classification. This permits for design of treatment measures such as sub-surface drainage for clay sections.

For a rational approach to pavement design, the most important characteristic of the subgrade is its elastic modulus. However, the measurement of this modulus requires complicated and time-consuming tests.

It has been proven that there is a good correlation between the elastic modulus and the California Bearing Ratio (CBR). Various empirical relations for CBR to elastic modulus have been developed for different soil types. Since the CBR test is a fairly easy and widely used test, it has been decided to retain it as the quantitative means of evaluating the subgrade bearing strength.

4.2 Determination of Subgrade Strength

4.2.1 Recommended Subgrade CBR Test Procedure

The actual strength of the subgrade and its actual CBR depend on the type of material, its density, and its moisture content.

For each type of material, it is therefore necessary to determine the relative compaction that should be obtained in-situ and the maximum moisture content likely to occur in the subgrade.

In order to obtain a complete knowledge of the relationship between density, moisture content and CBR, a '6 point' CBR test should be carried out on a representative sample of each type of subgrade material encountered. The tests are conducted in the following way:

> The material shall be compacted at 3 different levels of compaction. (90% MDD, 95% MDD, and 100% MDD) in accordance with AASHTO T 99. The samples shall be moulded at the moisture content which is expected at the time of in-situ compaction (in general, at the Optimum Moisture Content). At each level of compaction, one CBR shall be measured on one unsoaked and one soaked specimen. The time of soaking will depend on the anticipated subgrade conditions. The amount of water absorbed during soaking, and the eventual swell shall also be measured.

The above method enables an estimate to be made of the subgrade CBR at different densities and thus helps in deciding the relative compaction required. It also indicates the loss of strength which soaking may cause. A full particle size analysis should also be done on each representative sample.

4.2.2 Classes of Subgrade Bearing Strength

The California Bearing Ratio (CBR) is dependent on the type of soil, its density, and its moisture content. Thus, the classification of the subgrade material is based on the soaked California Bearing Ratio (CBR) at a representative density. The subgrade strength for design shall be assigned to one of six bearing strength classes as defined in Table 4.1.

Table 4.1 Subgrade Strength Classes

Subgrade Class	CBR Range (%)	Median CBR (%)
S1	2-5	3.5
S2	5-10	7.5
S3	7-13	10
S4	10-18	14
S5	15-30	22.5
S6	30-60	45

The above CBR ranges correspond to the results actually obtained on materials of the same type along sections of road considered homogeneous. They reflect both the variations of the characteristics of the soil which inevitably occur, even at small intervals, and the normal scatter of test results.

The following points should be noted:

1. It will be noted that the subgrade categories overlap. For any one section of road, the average CBR should be higher or equal to the median of the subgrade class selected for design and no individual result shall be below the lowest value of the range for that subgrade class (see Figure 4.1). Where the subgrade CBR values are very variable the designer should balance the cost of having very short uniform sections of different subgrade classes against a conservative design based on the worst condition encountered over longer sections.

2. No provision has been made for subgrade of CBRs below 2 % because it is technically and economically unviable to lay a pavement on soils of such poor bearing capacity. Such weak soils are saturated expansive clays, saturated fine silts or compressible (swampy) soils, e.g. mud, soft clay, etc. moreover, the measurement of bearing strength of such soils is most uncertain and CBR values below 2 % have little significance. They should be dealt with as described in Chapter 3.

3. Pavements to be constructed on subgrade class S1 (CBR 2-5) shall require improvement as described in Chapter 6 and 7 because of their poor bearing capacity. Such weak soils are saturated expansive clays, saturated fine silts or compressible (swampy) soils e.g., mud, soft clay, etc. Moreover, the measurement of the bearing strength of such soft soils is most uncertain and CBR's below 3 are of little significance.

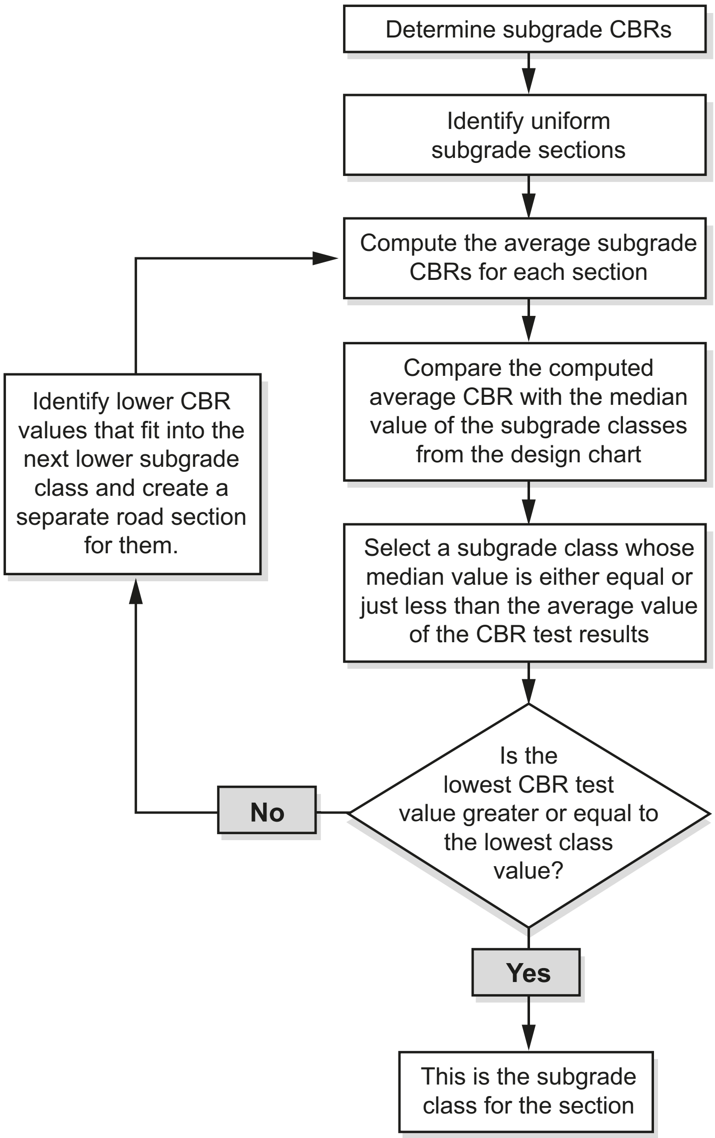
4. The use of subgrade class S2 soils as direct support for the pavement is only permissible for unpaved roads. All other pavements shall require a minimum equivalent subgrade class of S3. Capping options for improving subgrades are provided in Chapter 4.

5. The CBR range of subgrade class S5 is fairly wide. This is because the difference in the pavement thickness required is comparatively small when the subgrade bearing strength varies from the lower to the upper limit of this class.

6. Class S6 covers all subgrade materials having a CBR in the range 30-60, and which comply with the plasticity requirements for natural materials for sub-base. In such cases, no sub-base is required for low and medium traffic pavement (Traffic loading below 10 million cumulative standard axles). No class of higher bearing capacity has been considered as such subgrades are extremely rare.

The procedure for determining the subgrade class is shown in Figure 4.1. For any one section of a road the average (CBR should be higher or equal to the mean of the subgrade class selected for design, and no individual result shall be below the lowest value of the range for that subgrade class). Where the subgrade CBR values are highly variable the designer should balance the cost of having very short sections of different subgrade categories against a conservative design taking account of the worst conditions encountered over longer sections.

Figure 4.1 Determining the Subgrade Bearing Strength Class



4.2.3 Classification of the Most Common Kenyan Subgrade Materials

Typical subgrade materials encountered in Kenya are presented in Table 4.2. The bearing strength classes presented are tested at standard compaction effort and are only indicative of the materials strength. The designer shall obtain CBR values through laboratory tests. General engineering classification of the soils is given in RDM Volume 3 Part 1.

Table 4.2 Common Kenyan Subgrade Materials

Subgrade Class	Subgrade Class (100 % MDD ASHTO T99)	
	After 4 days soak	At OMC
Black cotton soils	S1	S5
Micaceous silts (decomposed rock)	S1	S3
Other eluvial silts (decomposed rock)	S2	S4
Red friable clays	S3	S5
Sandy clays on volcanics	S3 or S4	S5
Ash and pumice soils*	S3 or S4	S5
Silty loams on gneiss and granite	S4	S5
Calcareous sandy soils	S4	S5
Sandy clays on basement	S4	S5
Clayey sands on basement	S4 or S5	S5 or S6
Dune sands	S4	S4 or S5
Coastal sands	S4	S5
Weathered lava	S4 or S5	S5 or S6
Quartzitic gravels	S4 - S6	S5 or S6
Soft (weathered) tuffs	S4 - S6	S5 or S6
Calcareous gravels	S4 - S6	S5 or S6
Lateritic gravels	S5 or S6	S6
Coral gravels	S5 or S6	S6

Note: *Some of the ash and pumice soils have a very low maximum dry density and a lower Young's Modulus than might be expected from the measured CBR values. Such soils (Standard Compaction MDD less than 1.4 Mg/m^3) cannot be classified for pavement design purposes based on CBR only. Particle size distribution and aggregate crushing value play a very important role in their use. They should at least conform to the material class GCS-E (see Chapter 8.9) before they can be used as capping or in pavement layers.*

4.3 Determination of Subgrade Class for Road Section

4.3.1 Determination of Homogeneous Sections

The first step in determining uniform sections is to undertake a soil classification as described in RDM Volume 3 Part 1. The uniform sections should consist of similar soil groups e.g., clays, silts, sands, gravel or at a broader level such as coarse-grained soils, and fine-grained. These uniform sections based on soil classification to provide additional measures such as provision of sub-surface drainage in the sections containing silt or clay, appropriate cut slopes, subgrade modification treatment lengths, etc.

The next stage is the determination of uniform sections based on the subgrade strength. This is for the purpose of foundation design. The sections should not be too short that it presents construction challenges. If this is the case, then soil replacement should be considered before defining uniform sections. Following this, then the cumulative sums (CUSUM) method should be used to determine homogeneous sections and the respective subgrade classes. This involves:

1. Using a spreadsheet, listing all the chainages at which the subgrade samples were taken in one column. In the adjacent column listing the corresponding CBR values for each chainage. At each chainage (test pit), the CBR value used should be the minimum CBR from each test pit; or if the reduced level at which the design vertical road profile cuts through the test pit is known, then the weighted average of the strength of all profiles below the reduced level to a maximum depth of 0.5 m should be used. The vertical profile is taken as the grade level.
2. Computing the mean of all the CBR values for the entire road.

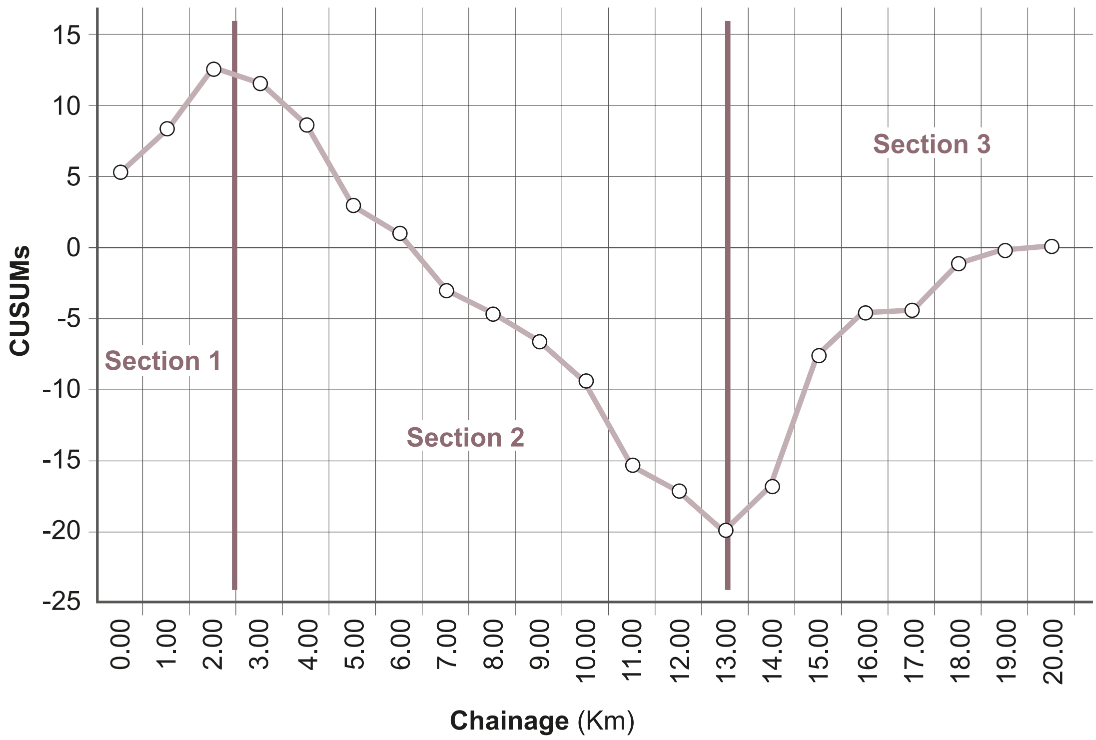
3. In the third column of the spreadsheet, subtracting the mean from the CBR values for each chainage.
4. In the fourth column of the spreadsheet, cumulatively summing (CUSUMs) the values in the third column.
5. Plotting the cumulative sums in the fourth column of the spreadsheet against the chainage.
6. By visual inspection of the graph, identifying points of inflection. These represent changes in subgrade strength, hence uniform sections, as shown in Figure 4.2.
7. Determining the average of the CBR test results under each homogeneous sections and the CBR bearing strength class of the section shall then be determined as described in section 4.2.2 and illustrated in Figure 4.1.

An illustration of the procedure for determining uniform subgrade sections by the method of CUSUMs is shown in Table 4.3 and Figure 4.2.

Table 4.3 Illustration of the Use of CUSUMs

Chainage (Km)	**Subgrade CBR**	**Mean CBR** Measured CBR	**CUSUM**	**Mean CBR** for the uniform section	**Minimum CBR** for the uniform section	**Subgrade Class** of the uniform section
0.00	12	5	5	14	12	S4
1.00	14	3	8			
2.00	13	4	12			
3.00	18	-1	12			
4.00	20	-3	9	20	19	S4
5.00	23	-6	3			
6.00	19	-2	1			
7.00	21	-4	-3			
8.00	19	-2	-5			
9.00	19	-2	-7			
10.00	20	-3	-9			
11.00	23	-6	-15			
12.00	19	-2	-17			
13.00	20	-3	-20			
14.00	14	3	-17	14	8	S3
15.00	8	9	-8			
16.00	14	3	-5			
17.00	17	0	-4			
18.00	14	3	-1			
19.00	16	1	0			
20.00	17	0	0			
Mean	17					

Figure 4.2 Example of Determination of Uniform Sections Using CUSUMs



4.3.2 Classification of Alignment Soils

The survey report from the RDM Volume 3 Part 1 will have information on subgrade engineering classification of alignment soils, for example, silty sand, clayey gravel, etc determined through laboratory tests. It will then be combined with laboratory test results for the strength (CBR). The report should contain a table that shows the chainage of each sampling point, the soil characteristics at each chainage, and the strength. Using these reports, the uniform subgrade sections should be determined as discussed in section 4.3.1, and reported accordingly. This output should then be used to design the foundation as discussed in Chapter 7.

5 Earthworks

5.1 General

The road cuts, embankment heights and fills are determined from the topography along the alignment. These data are also required for the design of horizontal and vertical curves. Therefore, topographical surveys are important for the entire geometric design. These will include conventional longitudinal and transverse level surveys along the entire length of the road at appropriate intervals – usually 20 m in the longitudinal direction on the proposed centreline and at 2 m interval offsets on each side of the centreline up to the edge of the right-of-way. The data should preferably be reported in a format that can be inserted directly into any geometric design software being used for the project. During this survey, temporary benchmarks must be fixed and referenced.

Topography data obtained by photogrammetry can be complete and accurate, with features such as fences, walls, utility poles, and existing improvements, in addition to elevation contours and spot elevations. These data will be of great use to the designer in determining road reserve restrictions, swamp or rock locations, and the need for relocating utility lines or other physical features that affect the design.

The topography data should be checked for completeness against other maps available to the designer. Topography data obtained by field ground surveys will generally be collected electronically for downloading to appropriate computer-aided design software.

5.2 Cuttings

5.2.1 General

Wherever a cutting is required, consideration needs to be given to the following factors that will affect its design and cost:

1. Type of material to be excavated.
2. Volume and position of the different materials.
3. Level and flow of water table and springs.
4. Stability of slopes.
5. Drainage and protection against erosion.

Any cutting to a depth of more than 5 m requires a specific study, including boreholes or test pits down to the formation level.

5.2.2 Type of Material to be Excavated

The type of material to be excavated governs the construction methods, the use to which the material can be put, its suitability as subgrade material and slopes that can be safely adopted.

From both economic and technical points of view, the respective volumes of rock, rippable and normal material occurring in each deep cut shall be determined with reasonable accuracy. Dynamic penetration and seismic tests may usefully supplement boreholes for this purpose.

At the design stage, it is sometimes not easy to accurately differentiate rock from rippable material or rippable from normal material. In particular, the occurrence of residual boulders and especially their volumes are, in some cases, difficult to ascertain.

Table 5.1 can be used as a guide to determine the excavation characteristics of rock based on rock hardness and strength.

Table 5.1 Rock Excavation Characteristics

Rock hardness description	Unconfined compressive strength (MPa)	Excavation characteristics
Very soft	1.7 – 3.0	Easy ripping
Soft	3.0 – 10	Hard ripping
Hard	10 – 20	Very hard ripping
Very hard	20 – 70	Extremely hard ripping or blasting
Extremely hard	>70	Blasting

Tropical weathering generally results in the occurrence of two types of materials:

1. Residual (or alluvial) soils.
2. Weathered rock.

The depth and degree of weathering are often variable, and the properties of the residual materials may be quite different even within the same horizon over a short distance. In addition, the pattern of the residual materials is often complicated by the processes of leaching of silica and accumulation of iron and aluminium sesquioxides, known as laterization.

The depth of bedrock is clearly important not only because of its effect on the cost of the cuttings, but also because the presence of rock can provide a layer on which a perched water table can exist. Depending on the type of rock and its structure, springs may also be a problem.

The quality of cutting should be assessed for possible reuse as fill material, capping layers, or even sub-base layers on other sections of the road. The specifications are included in section 8.9.

5.2.3 Water Table and Springs

A water table may be permanent, seasonal, or perched. In any case, its presence, and its characteristics (level, flow of water, etc.) must be determined. Presence of water may affect the method of excavation and the stability of the cut slopes as well as the drainage system required. Similarly, the likelihood of springs occurring within the cut should be assessed.

5.2.4 Determination of the Angle of Slope

The design of the slope angle of a cut is a compromise between the following requirements:

1. Stability.
2. Erosion control.
3. Appearance and visibility.
4. Need of fill material.
5. Minimum cost.

5.2.4.1 Stability

The stability of cuttings is highly dependent on the height of the slope. For cuts less than 6 m deep where there is no water seeping out of the cut face and no external loads, the slope batters presented in Table 5.2 shall be used as a guide.

Table 5.2 Typical Slope Batters of Excavated Slopes

Material	Slope Batters (Vertical: Horizontal)
Cohesionless sands	1:2
Silty sands - silts	1:1
Eluvial soils (red friable clays)	1.5:1 if the depth of cut is < 4 m 1.5:1 if the depth of cut is > 4 m
Weathered rock	2:1 to 4:1
Sound rock	5:1 to 10:1

Notes

1. *The relic structure of the parent rock may dominate the stability in alluvial soils.*

2. *Safe slopes in schistose or stratified rock chiefly depend on the orientation of the foliation or strata. Similarly in rocks with a strong pattern of jointing, the orientation and spacing of the discontinuities may well control the safe slope angle.*

Any cutting to a depth of more than 6 m shall require geotechnical investigations, including boreholes down to formation level and associated testing. A geotechnical design (See RDM 4.3), by a geotechnical engineer, to determine the factor of safety against sliding or rotation considering the material strength parameters, the slope geometry, the location of any water table and the presence of any external loading on the slope.

5.2.4.2 Erosion Control

This is sometimes a difficult requirement to reconcile with stability. For instance, in cohesive materials, the erosion is less extensive for very steep slopes than for slopes at about 1:1 and it is therefore desirable to keep the slopes as steep as possible, unless the slope is protected.

Slope erodibility is controlled by the grades and type of soil. Generally, guidance on satisfactory slope angles from the points of view of both stability and erosion resistance can be obtained from a survey of other cuttings and natural steep slopes in the vicinity. In the red friable clays, which are very common in Kenya, a cut slope of 1:1 would be extensively eroded where its height exceeds 4–5 m.

For slopes prone to erosion or where erosion is foreseen the following options shall be considered:

1. As a guide, topsoil thickness of a minimum of 75 mm shall be used. Thicknesses greater than 100 mm could result in gullying.
2. Grassing and planting shall be considered for slopes greater than 1V in 2H. Preferably, planting should be done at the beginning of the first rainy period. Bioengineering shall be considered suitable for slip surfaces of 1–2 m. Local native species for the region should be used. The choice of plant and grass species shall be dependent on the type of roots (deep or spreading) and site conditions such as moisture, permeability, temperature, stoniness, and nutrients in the soil, among others. Once the vegetation has been established the overall slope stability and erosion resistance increase.
3. Benching of slopes to reduce concentrated runoff. The bench width shall vary between 2–4 m to allow for maintenance requirements, and rockfalls where applicable. Benches shall always be sloped downwards out of the cut face at about 10–15 %, and a minimum depth of 0.3 m to prevent water ponding. The bench height is dependent on the runoff, the type of material and the overall risk associated with the slope. As a guide, the bench heights in Table 5.3 shall be considered during design.
4. Small check dams shall be constructed on steep slopes and where long erosion gullies are foreseen.

Table 5.3 Typical Benching Heights

Slope	Vertical Height Between Benches
1V: 4H	20 m
1V: 3H	15 – 20 m
1V: 2H	10 – 15 m
1V: 1H	5 – 10 m

5.2.4.3 Other Factors

Partly for aesthetic and safety reasons, a low angle slope is normally considered more desirable than a near-vertical one, even if other factors will allow this latter course.

The need for, or the surplus of, fill material will also have an influence on slope angles.

In deep cuttings, where the pavement is laid shortly after completion of the cutting, consideration should be given to heave.

5.3 Fill Embankments

5.3.1 General

Wherever an embankment is required, consideration needs to be given to the following factors that will affect its design and cost:

1. Foundation conditions.
2. Settlement.
3. Method and rate of construction.
4. Stability of slopes.
5. Acceptable fill material.

5.3.2 Foundation Conditions and Settlement

Embankment stability relies greatly on the bearing capacity of the underlying ground, and the nature and proximity of the groundwater table. Most cohesionless soils exhibit good bearing capacity and low compressibility. Settlement in cohesionless soils is generally negligible and occurs rapidly during the placement of the fill. However, their liquefaction potential should be considered for projects in seismically active zones.

Embankments constructed on swamps with soft ground, such as soft clays, silts and organic materials, including peat, undergo consolidation settlement. Swamps shall be classified as seasonal or permanent swamps. The type, number, length, and corresponding chainage of each swamp shall be recorded during site investigations. The length of permanent swamps shall have to be confirmed during the wet season as it may vary.

The foundation conditions beneath such embankments therefore require special attention to avoid shear failures and excessive settlements. Detailed investigations should be conducted to determine the most suitable construction method, the rate of construction and any special precautions required, for embankments constructed on compressible soils.

It is essential to use ground improvement methods which ensure either the removal of the soft material or its substantial consolidation before the pavement is completed. Cost-benefit analysis of the various methods which are technically feasible shall be undertaken during the design before choosing a method. The following ground improvement methods can be considered during design:

1. **Removal and replacement:** This involves the excavation of the compressible soil from below the embankment and replacing it with higher quality less compressible soil. Usually, the excavation should extend to at least the toe of the embankment to increase the stability of the embankment. Removal and replacement is only justified under certain conditions because of the high costs associated with excavating and disposing of unsuitable soils and the difficulty of excavating below the water table. Displacement using rockfill may be used for material below the water table. Some of the conditions where removal and replacement shall be deemed feasible include the following:
   - i. The area requiring excavation is not wide.
   - ii. The unsuitable soils are near the ground surface and do not extend very deeply (removal of unsuitable material beyond the depth of 3 m is not normally economically feasible).
   - iii. Temporary dewatering is not required to support or facilitate the excavation.
   - iv. The unsuitable soils can be dumped on-site or can be disposed of safely elsewhere close by.
   - v. Suitable fill materials are readily available to replace the volume of unsuitable soils.
2. **Allowing for consolidation**: This involves pre-loading (surcharging) with higher embankment fill, installation of prefabricated vertical sand drains to accelerate settlement or a combination of pre-loading and sand drains. Normally 90 per cent consolidation before placing pavement layers is considered adequate.

If the consolidation solution is chosen the rate of dissipation of construction pore pressures in the soft, saturated foundation material must be investigated and a suitable construction rate fixed. This is most important, especially if a high pre-loading embankment is proposed, in order to obviate a deep-seated shear failure during construction. Even if the proposed embankment is only a few metres high a full geotechnic al investigation is necessary to determine the magnitude and rate of settlement, and the likely pore pressures to be developed during construction. Where the estimate of this last parameter is not very reliable (as is usually the case) peizometers can be install ed in the foundation material. This will allow faster safe rates of construction if the forecasts have been pessimistic and will prevent stability failure if they have been optimistic.

5.3.3 Geotechnical Design Considerations

Embankment fills over swamps are frequently stronger and stiffer than their foundations therefore the peak strengths of the embankment and the foundation soils do not mobilise simultaneously. Thus, an embankment may deform as the foundation fails under the weight of the embankment with the possibility of progressive failure because of stress strain incompatibility between the embankment and its foundation. Hence, during design attention should be given to the internal stability of the entire embankment foundation interface rather than the embankment or the foundation soils alone.

Consolidation parameters play a crucial role in geotechnical engineering, particularly in embankment design. Consolidation refers to the process by which soil undergoes volume reduction due to the expulsion of water from its void spaces under sustained loading. The primary consolidation parameters include the coefficient of consolidation (C_v), coefficient of volume compressibility (m_v), and the compression index (C_c). They are defined as follows:

1. **Coefficient of Consolidation (C_v):** The coefficient of consolidation denoted as cv, represents the rate at which soil consolidates under load. It is determined through laboratory tests such as the oedometer test. Higher values of C_v indicate faster consolidation, implying that the soil achieves stability more rapidly.
2. **Coefficient of Volume Compressibility (m_v):** The coefficient of volume compressibility, denoted as m_v, measures the change in volume per unit change in effective stress. Similar to C_v, it is determined through laboratory tests, often obtained from oedometer tests. A lower value of m_v indicates lower compressibility, suggesting that the soil is less prone to volume reduction under load.

3. **Compression Index (C_c):** The compression index denoted as C_c, is the slope of the virgin compression line on the *e-log p* curve. Determined from the oedometer test, C_c provides insights into the compressibility characteristics of soil. A higher C_c implies greater compressibility, reflecting the potential for greater settlement under loads.

The conventional oedometer consolidation tests with specimens cut from undisturbed samples as discs in the horizontal plane, normally give accurate predictions of the amount of settlement that can be expected for a layer of soft saturated clay loaded by an embankment. However, the time of settlement predicted by this method is usually much longer than in practice. This is because in most normally consolidated clays the drainage path in the horizontal direction is usually many times more permeable than it is in the vertical direction. Oedometer tests with specimens cut from undisturbed samples in the vertical plane will give an accurate prediction of the time of settlement under an embankment loading.

Generally, settlement in the range of 30 to 60 mm throughout the design life of the road shall be tolerable provided that it is uniform, occurs slowly, and does not take place adjacent to a pile supported bridge.

If the consolidation solution is chosen the rate of dissipation of construction pore pressures in the soft saturated foundation material must be investigated and a suitable construction rate fixed. This is important, especially if a high pre-loading embankment is proposed, to avoid a deep-seated shear failure during construction. Where the estimate of the magnitude and rate of settlement is not very reliable (as is usually the case) piezometers can be installed in the foundation material. This will allow faster safer rates of construction if the forecasts have been pessimistic and will prevent stability failure if they have been optimistic.

The design of embankments of heights greater than 3 metres should be undertaken by a geotechnical engineer. The design steps are as follows:

1. **Geometry and Loading Conditions:** The geometric parameters required are the height and length of the embankment, the width of the crest (shoulder break-to shoulder break), and the slide slope angle. The loading conditions include any surcharges and any temporary or dynamic loads. The construction rate should also be included because the gain in shear strength is directly affected by the placement of the embankments.
2. **Soil Profile and Engineering Properties:** The subsurface stratigraphy should be determined, including soil layering and groundwater table location. The testing should include basic classification testing (RDM 3.1). The shear strength and consolidation properties should be from laboratory testing.
3. **Embankment Fill Engineering Properties:** The engineering properties of the fill (borrow) material should be determined, including basic classification testing (RDM 3.1), moisture-density relationship, shear strength, and chemical properties. A drainage media (e.g., free-draining granular materials, non-woven geotextiles, etc.) should be placed at the interface between the existing subgrade and the embankment fill to permit drainage of water. Above this drainage media, normal backfill materials may be placed.
4. **Bearing Capacity Check:** The bearing capacity of the subgrade soils can be checked.
5. **Rotational Shear Stability Check:** Perform a rotational slip surface analysis on the embankment to determine the critical failure surface and the resistance factor against local shear instability.
6. **Sliding Block Stability Check:** Perform a sliding block analysis. If the calculated resistance factor is less than required, then reinforcement is not required. If the resistance factor is inadequate, then reinforcement is required.
7. **Estimate Magnitude and Rate of Embankment Settlement:** The magnitude and rate of immediate settlement, primary settlement and secondary embankment settlement should be computed. These require the following design inputs:

i. Total stress and effective stress strength parameters.
ii. Unit weight.
iii. Compression indexes (primary, secondary and recompression).
iv. Coefficient of consolidation.

8. **Establish Construction Sequence and Procedures:** The construction sequence and procedures should be established. Proper placement and performance of the geosynthetic is highly influenced by the construction sequence and procedure. The sequence and procedure should be as clear and concise as possible to prevent misunderstandings during construction.
9. **Establish Construction Observation Requirements:** Since implemented construction procedures are crucial to the success of reinforced embankments on very soft foundations, competent and professional construction inspection is absolutely essential. Field personnel must be properly trained to observe every phase of the construction and to ensure that the specified material is delivered to the project and the specified construction sequence is explicitly followed. Instrumentation requirements should also be established. As typical instruments include piezometers, settlement points, surface survey points and slope inclinometers. Part of the instrumentation requirements is establishing how often the measurements will be obtained.

5.3.4 Stability of Slopes

Embankments less than 3 m in height in areas having stable ground generally do not require a detailed geotechnical investigation and analysis to determine their slopes. The slopes, based on past experience in the same region and on engineering judgment, given in (Table 5.4) are recommended for cohesionless sands and other materials.

Table 5.4 Typical Slope Batter

Material	Embankment Height (h)	Recommend Slope (Vertical: Horizontal)
Cohesionless sands	h ≤ 1 m	1:3
	h > 1 m	1:2
Other materials	h ≤ 1 m	1:3
	1 m < h < 3 m	1:2
	3 m < h < 10 m	1:1.5

A geotechnical analysis (see RDM 4.3) shall be undertaken to determine the stability of embankments higher than 3 m or those of lesser height but founded on soft, wet materials using the cohesion and angle of friction of the embankment fill and founding materials.

Typically soil slopes shall not exceed 400 unless reinforcement or a retaining wall is used.

The moisture variation zone should be contained within the side slopes. That is, the slope angle chosen should be such that the lateral distance from the toe of the embankment to the outer edge of the shoulder should be at least 2 metres.

Embankments higher than 3 m or those of lesser height but founded on soft, wet materials should be analysed individually using appropriate geotechnical methods (see RDM Volume 4 Part 3).

Where land and fill material are readily available, modification of the embankment geometry through reduction of the slope angle or the construction of counterweight berms may be considered to improve the stability of the embankment.

Curbs, or an alternative method of controlling runoff along the embankment slope should be specified for all pavement systems on compacted embankments to improve stability and reduce maintenance costs.

The provision of riprap at the embankment toe where scouring of the toe is foreseen for is recommended for additional slope protection. This is also important due to climate resilience requirements.

The safety factors for overall stability under static conditions shall be as follows:

1. All embankments not supporting, or potentially impacting structures shall have a minimum safety factor of 1.25.
2. Embankments supporting or potentially impacting non-critical structures shall have a safety factor of 1.3.
3. All Bridge approach embankments and embankments supporting critical structures shall have a safety factor of 1.5.
4. Critical structures are those for which failure would result in a life-threatening safety hazard for the public, or for which failure and subsequent replacement or repair would result to high financial burden.

5.3.5 Fill in Hard Material

Hard material refers to a material of UCS greater than 7.5 MPa (tested on a core of 100 mm diameter X 200 mm length) or Mohs hardness is equal to or greater than 5.

Hard material shall be material which cannot be ripped to an average depth of rip greater than 300 mm by a track type craw1er tractor complying with the following:

1. In good order complete with all equipment and accessories as supplied;
2. Rated 300 BHP flywheel power or over;
3. With an operating weight of not less than 37.2 tonnes;
4. Equipped with a hydraulically operated single tine ripper compatible with the tractor used; and,
5. Table 5.1 can be used as a guide to determine the excavation characteristics of rock based on rock hardness and strength.

Where it is impractical to prove hard material by the above method then the quantity of hard material shall be determined by the Engineer based UCS tests on core samples of 100 mm diameter x 200 mm length and the classification in Table 5.1. Based on this method, hard material shall be defined as a material of UCS greater than 7.5 MPa or Hard material shall be defined as material whose minimum and average unconfined compressive strength determined from a sample of six cores, exceeds 7.5 MPa and 10 MPa.

Hard material shall not be placed within 600 mm of the formation level in embankments and shall be removed to a depth of 300 mm or as otherwise instructed by the Engineer below formation level in cuttings.

Any fill material meeting these requirements and is used to form an embankment shall be classified as 'fill in hard material'. These consist of rocks, recycled concrete material, building waste, etc.

This shall often require rolling to break down into sizes less than 100 mm as the largest dimension. Blinding using finer material to fill in any spaces between larger particles is required before placing any pavement layers on top.

5.3.6 Fill in Soft Material

Fill material will generally be obtained from cuttings. If the material obtained from this source is insufficient or unsuitable, extra material shall be obtained from borrow areas.

Any fill material not classified as 'hard' shall be classified as 'soft'.

The following materials are generally unsuitable for the construction of fills:

1. All material containing more than 5% by weight of organic matter (such as topsoil, material from swamps, mud, logs, stumps and perishable material).
2. All material having CBR at 100% MDD (AASHTO T99) and after 4 days soak of less than 2%.
3. All material with a swell of more than 3% (such as black cotton soil).
4. All clay of plasticity index exceeding 50 (However, some red friable clays having a plasticity index over 50 may successfully be used). It is known that alluvial soils (red friable clays), when compacted in embankments to a greater density than they are found in-situ, develop considerable shrinkage and suction forces associated with seasonal rainfall and dry periods, together with the root system of the grass planted on embankments.

   These forces are large and can cause longitudinal cracks to repeatedly be formed through the surface of any pavement containing rigid or semi-rigid layers.

   It is suggested that if no other earthwork materials are available then a fully flexible pavement on these embankments will be most suitable. If for reasons of traffic category, a pavement incorporating rigid or semi-rigid layers is necessary (asphalt, bituminous macadam, lean concrete, concrete cement or lime stabilised or improved gravels, it is suggested that the problem may be overcome by incorporating a 'slippage' layer which will stop the cracks being transferred through the pavement. A suitable 'slippage' layer will comprise the placing of a layer of polythene sheeting at the top of the subgrade earthworks and laying a thin 'lower sub-base" of sand or crushed dust, before the sub-base.
5. All material having a moisture content greater than 105 per cent of the Optimum Moisture Content (Standard Compaction).

The best materials either from cuttings or from borrow areas, should be reserved for the upper layers of fill.

5.3.7 Placing and Compaction of Fill

5.3.7.1 Construction on Near Level Ground

Generally, all soft and organic material shall be removed, and hollows shall be filled, to obtain a uniform surface to receive the fill. Any backfilling required to obtain the uniform surface shall be compacted to a dry density of at least 100 % MDD (AASHTO T99).

5.3.7.2 Construction on Slopes

Where the slope of the existing ground is greater than 1 (vertical) to 3 (horizontal), horizontal benches in steps not less than 3 m wide shall be cut into the existing ground. Immediately on completion of cutting the benches, the whole of the area to receive the fill shall be compacted to 100 % MDD (AASHTO T99) down to a depth of 150 mm. The time between preparing the area and placing the fill must be kept to a minimum.

5.3.7.3 Compaction of Fill Material

Materials other than rockfill shall normally be placed in layers of compacted thickness not exceeding 150 mm. Thicker layers (not exceeding 250 mm) may be permitted only after trial sections have proved that the required compaction may readily be obtained over their full depth.

The minimum layer thickness shall be twice the maximum particle size of the material.

Normally, the layers of fill material shall be compacted throughout to a dry density of at least 100 % MDD (AASHTO T99), except for the upper 300 mm of the subgrade, which shall be compacted to a dry density of at least 95 % MDD (AASHTO T180).

For very high fills, higher compaction may be required to reduce settlement.

The moisture content of the material shall be adjusted so that the above specified minimum compaction are obtained. Moisture contents well below the Optimum Moisture Content may be accepted, provided that the compaction equipment and method are such that the required compaction is achieved. Especially in arid areas, dry compaction may offer substantial savings or maybe the only practical solution.

Dry compaction by vibratory rollers operating at the proper frequency and amplitude seems to give fairly good results on some non-plastic materials. However, experience is still very limited, and the development of dry compaction requires further experiments and research.

It is strongly recommended that the moisture content at the time of compaction does not exceed 105 % of the Optimum Moisture Content. This applies particularly to silty and clayey materials, which are prone to shrinkage and loss of strength, resulting from excessive moisture contents.

Normal laboratory compaction tests cannot be accurately carried out on materials containing a high proportion (say more than 25 %) of particles greater than 40 mm size. On such coarse material, the minimum dry density required, and the suitable moisture content shall be determined from site compaction trials.

Where fairly homogeneous materials are used, the compaction requirements may consist of a method specification with the following parameters being fixed:

1. The maximum thickness of the compacted layer.
2. The characteristics of the compacting equipment.
3. The number of passes of each roller.
4. The permissible range of moisture content, all as determined from full-scale compaction trials.

5.3.8 Dump Rock

Dump rock shall be natural or quarry stones having minimum core UCS of 7.5 MPa and shall comply with the requirements in Table 5.5. Dump rock may be placed in layers not thicker than 1 metre in one operation. Each boulder shall not be larger than two-thirds (usually not larger than 600 mm diameter) of the compacted thickness. It is dumped in soft uncompactable soils as initial step in embankment construction in swampy soils. Where impermeable layers of fill are required, dump rock shall not be used. Where applicable a geosynthetic may be used for separation, followed by the placement of fill material and/or improved subgrade layers.

Table 5.5 Requirements for Dump Rock in Fill

Material Properties	**Material Class** (Dump Rock: Min UCS 7.5 MPa)
Maximum particle size	Two thirds of compacted layer thickness.
Maximum layer thickness	1 m placed in one operation.

Note: *The content of fines shall be sufficiently low so that the larger particles rest against each other when placed in earthworks layers. The Engineer's decision shall be the final in cases where doubt or disagreement exists whether the material can be classified as DR.*

5.3.9 Rockfill

This is rock material of such particle size that the material can only be placed in layers of compacted thickness exceeding 300 mm. Boulders with volumes greater than 0.2 m^3 (600 mm size) are not normally used. This material should not be placed within 600 mm of the formation level. The rock shall have a minimum UCS of 7.5 MPa.

Where rockfill is used it should be placed in the bottom of the embankment. The largest sizes of rock shall be placed in layers of maximum compacted thicknesses of 1 m. The interstices shall then be filled with smaller rocks, spalls and approved finer material. The whole layer shall be compacted until the interstices are completely filled or until the required settlement is obtained. Heavy vibratory rollers are generally the most suitable machines for compacting rockfill.

It is most important that the specified compaction is achieved over the full width of the embankment. Loose material left on the slopes may absorb water and may endanger the stability of the slopes.

6 Problematic Soils

6.1 General

This chapter addresses problematic soils which pose challenges during construction and performance of civil engineering structures founded on soils. They include expansive, collapsible, dispersive, low strength, low density, saline and high organic content soils. These soils need to be identified at design stage and appropriate interventions measures included in the design documents to enable proper performance over the envisaged service life.

6.2 Problems Associated with Expansive Clays

6.2.1 Geotechnical Characteristics of Expansive Clays

Some clay soils undergo slow volume changes that occur independently of loading and are attributable to swelling or shrinkage. These volume changes can give rise to ground movements which can lead to cracking and breakup of the road they support. Furthermore, these volume changes may produce lateral displacements ('creep') if the side slopes are not gentle enough.

The principal cause of expansive clays is the presence of swelling clay minerals such as montmorillonite. Differences in the period and amount of precipitation and evapotranspiration are the principal factors influencing the swell-shrink response of a clay soil beneath the road pavement. Trees with high water demand may dry out clay causing shrinkage.

The potential for volume change in clay soil is governed by its initial moisture content, initial density or void ratio, its microstructure and the vertical stress, as well as the type and amount of clay minerals present. These clay minerals are responsible primarily for the intrinsic expansiveness whilst the change in moisture content or suction controls the actual amount of volume change, which a soil undergoes at a given applied pressure.

There are two modes of swelling in clay known as inter-crystalline and intracrystalline. Interparticle swelling takes place in any type of clay deposit irrespective of its mineralogical composition, and the process is reversible. In relatively dry clays the particles are held together by relict water under tension from capillary forces. Upon wetting, the capillary force is relaxed and the clay expands. In other words, inter-crystalline swelling takes place when the uptake of moisture is restricted to the external crystal surfaces and the void spaces between the crystals. Intracrystalline swelling, on the other hand, is characteristic of the smectite family of clay minerals, and montmorillonite in particular. The individual molecular layers, which make up a crystal of montmorillonite, are weakly bonded so that on wetting water enters not only between the crystals but also between the unit layers which comprise the crystals. Generally, kaolinite has the smallest swelling capacity of the clay minerals and nearly all of its swelling is of the interparticle type. Illite may swell up to 15% but intermixed illite and montmorillonite may swell some 60 to 100%. Swelling in calcium montmorillonite is very much less than in sodium variety, it ranges from about 50 to 100%. Swelling in sodium montmorillonite can amount to 2000% of the original volume, the clay then having formed a gel.

Because expansive clays normally possess extremely low permeabilities, moisture movement is slow and significant period of time may be involved in the swelling-shrinkage process. Accordingly, moderately expansive clays with a smaller potential to swell but with higher permeabilities than clays having a greater swell potential may swell more during a single wet season than more expansive clays.

Cemented and undisturbed expansive clay soils often have a high resistance to deformation and may be able to absorb significant amounts of swelling pressure. Therefore, remoulded expansive clays tend to swell more than their undisturbed counterparts. When the moisture content increase, expansion occurs and the bearing strength of the black cotton soil decreases dramatically. The CBR may be reduced to less than 2% if the soil becomes completely saturated.

6.2.2 Identification and Classification of Expansive Clays

Expansive clay subgrades may be identified in the field by observation of large cracks that appear on the land surface in the dry season and disappear in the wet season. After desiccation, the surface exhibits polygonal shrinkage cracks, which reflect the percentage of clay and possibly the presence of expandable clay minerals.

In the laboratory these soils are identified using index properties, bearing strength and swell tests. Experience has shown that the volume change behaviour correlates reasonably well with liquid limit, plasticity index, swell, and plasticity modulus (as shown in Table 6 1).

For instance, if the liquid limit is above 70%, then the material is often highly expansive and may not be suitable for fills. If the liquid limit is between 50% and 70%, then some treatment may be necessary to avoid distress. Similarly, the plasticity index is also a useful indicator; values below 15% indicate minimal problems and values greater than 45% indicate that the material must be treated or discarded.

The swell test is the second and most direct method for measurement of volume change, providing swell and swell pressure values. The swell defines deformation, while the swelling pressure is related to stress generated by volume change. The Oedometer swell test with minimum surcharge may be used to determine volume change. However, for practical design purposes, the CBR swell test may be used. Expansive clays will exhibit swell exceeding 2.5 %.

The investigations to identify and classify expansive soils according to their expansiveness are presented below:

1. Routine Investigations are those carried out during surveys of the project.
2. Extended Investigations include simple additional indicator testing in the laboratory when expansive soils are suspected.
3. In depth studies include specialised laboratory testing and are used when extended investigations show the occurrence of expansive soils, and the required countermeasures have high economic consequences.

Routine investigations are those analyses carried out during normal centreline soils surveys and site observations, as discussed in the first paragraph of this sub-subsection.

Extended investigations include simple additional laboratory tests to estimate expansiveness and shall be routinely employed where special measures against damage from expansive soils are proposed in the design. These include:

1. Shrinkage Limit (ASTM D4943-89).
2. PI_w>20%, where PI_w = Plasticity Index tested on fraction <425 µm, weighted for the sample's actual content of particles <425 µm as follows: $PI_w = PI$ x (% passing 425 µm).
3. Determining the activity, A_c of the soil sample. An activity value greater than 1.25 is indicative of active clays with a high potential for expansion.

   Activity = Plasticity Index / (% by weight finer than 2 µm). Inactive (good) and active (difficult) soils have A-values of less than 0.75 and more than 1.25, respectively. Further investigation of soils with values in between 0.75 and 1.25 will be at the discretion of the designer.
4. Classification of low/medium/high/very high expansiveness (Table 6.1).

The treatment principles shall be governed by the following criteria in Table 6.1 and Table 6.2 and presented diagrammatically in Figure 6.1.

Table 6.1 Classification of Expansive Soils

Expansive Classification	Liquid Limit %	Plasticity Index %	Plasticity Product[1]	Swell % in
Very high	> 70	> 45	> 3,200	> 5.0
High	> 70	> 45	2,200 – 3,200	2.5 – 5.0
Moderate	50 – 70	25 – 45	1,200 – 2,200	0.5 – 2.5
Low	< 50	< 25	< 1,200	< 0.5

Notes: *[1]PM = PI x % soil < 0.425 mm (425 micron). Soil at OMC compacted at 98% MDD with 4-day soak, 4.5 kg surcharge*

6.2.3 Susceptibility to Erosion

When dry, black cotton soils present a sand like texture (polyhedral segments formed by the agglomeration of clay, silt, and sometimes sand particles). In this state, they are prone to erosion to a much greater extent than that normally anticipated from their plasticity and clay content.

6.2.4 Design and Construction Procedures on Expansive Soils

6.2.4.1 General

The four common approaches for dealing with expansive clay subgrades are as follows:

1. Avoid the areas of expansive clays by realigning the road;
2. Excavate and replace the expansive clays with suitable material;
3. Chemical stabilisation of the expansive clay preferably with lime; and,
4. Minimise moisture changes and potential swell in the expansive clays.

The mitigation options for problems presented by expansive clays are presented in Table 6.2. The design engineer is advised to carry out a cost-benefit analysis before selecting a suitable approach.

Figure 6.1 Schematic of Embankments and Treatments on Expansive Soils

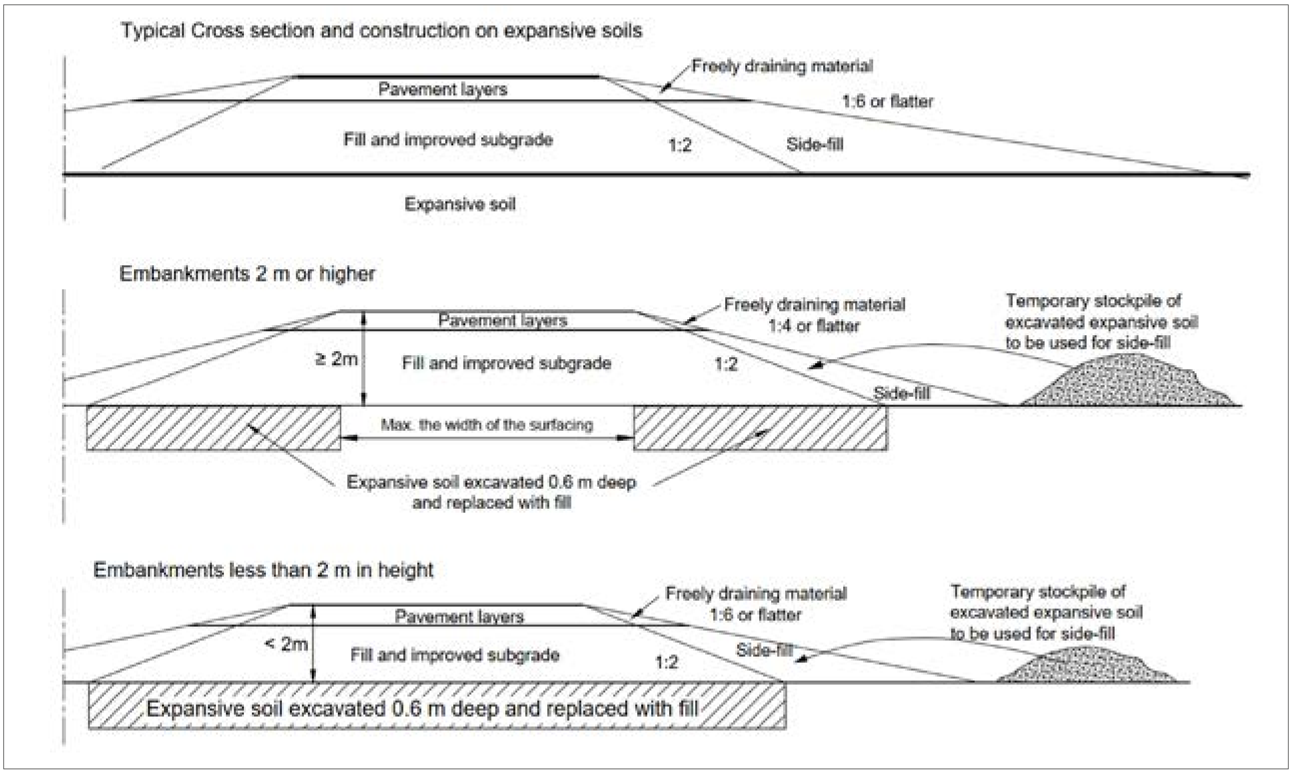


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Table 6.2 Summary of Mitigation Options for Road Construction on Expansive Clays

Options	Summary Description
1. Shift Road alignment	Re-align Road onto areas of non-problem soil.
2. Removal and replacement	For clays classified as 'High' or 'Very High' expansivity, dig out of the material and replace with inert or encapsulated material, Depth of dig out will be a function of the variation in swell potential and the depth of moisture content variation. An additional precaution is to treat with lime, any extra material that is not removed. Rockfill and sandbags (DONOU Technology for low volume roads) is useful in waterlogged areas prior to application of other soil.
3. Chemical stabilisation	Treatment of the expansive clays with hydrated lime or other chemical stabilisers including proprietary products approved by the Ministry
4. Minimise moisture changes and potential swell in the expansive clays	Placing of sufficiently high embankment of non-expansive material over expansive material such as to inhibit heave.
a. Confining expansive clays under improved subgrade and protective blanket.	For clays classified as 'Moderate' expansivity. Use of improved subgrade but is extended to include side-slopes and the toe of embankments. Cover material should ideally be a plastic gravelly soil. The road section should have sealed shoulders and flatter embankment side slopes (1:4 to 1:6). Rockfill and sandbags (DONOU Technology for low volume roads) is useful in waterlogged areas prior to the application of other soil.
b. Surcharge	Placing of sufficiently high embankment of non-expansive material over expansive material such as to inhibit heave.
5. Moisture control	Range of options: i. 2m wide sealed shoulders. ii. Impermeable full-width sub-base. iii. No vegetation allowed on shoulders. iv. 5% crossfall on shoulder. v. Drains to be lined, if unlined to be at least 4 m from embankment toe and shallow vi. Pre-wetting (2 - 3 months) to induce equilibrium moisture content before constructing the pavement. vii. Minimising or preventing moisture change using waterproofing membranes and/or vertical moisture barriers. The clay beneath culverts must be replaced with an inert material, all joints must be carefully sealed to avoid leakage and inlets and outlets must be well-graded to avoid ponding
6. Mechanical stabilisation	Mixing of potentially expansive material with inert material (e.g. sand/silt) to render the subsequent mass less potentially expansive.
7. Geosynthetic	Geomembranes used as impermeable barriers to vertical and horizontal moisture movement, although problems have been reported with placement, durability and possible hydrogenesis.

6.2.4.2 Realignment

This solution is possible only if the areas covered with expansive clays are of limited extent.

6.2.4.3 Excavation and Replacement

This procedure is simple and guarantees that the problems associated with expansive clays are eliminated. Its use is therefore recommended as much as possible. Backfill material is to be obtained from borrow pits.

However, this method is economically practicable only if suitable backfill material is available in the vicinity of the road. This is the case in most regions of Kenya where black cotton soils occur. Indeed, the thickness of black cotton generally does not exceed 1.00 - 1.20 m and suitable backfill material is found under the black cotton soils (e.g. decomposed phonolite in the Nairobi area).

In practice, it should be sufficient to excavate the expansive soil to a depth of about 1 m and cart it to spoil. Even if some expansive soil remains under the backfilling material, it should be adequately confined and protected from moisture changes.

It is recommended that backfill material be at least of S2 quality and impermeable enough not to act as a drain.

Embankments shall be constructed with suitable fill material (as specified in Section 5.3.6) obtained from borrow pits.

6.2.4.4 Treatment with Lime

Treatment of black clays with hydrated lime gives excellent results. The addition of 4 to 6 % of Lime is usually required and provides the following improvements:

1. Reduction of the plasticity index to less than 20.
2. Reduction of the swell to negligible values.
3. Increase of the CBR to a minimum of 10 (after 7 days cure) and 15 (after 28 days cure).
4. Modification of the particle size distribution (by agglomeration of the clay particles), the final grading being similar to that of a silt.

All these improvements render the treated clay easily workable, and it can be assumed that the treated clay will normally be of at least S4 quality.

This treatment is comparatively costly, however, because it is necessary to treat a substantial thickness of soil (minimum 300 mm compacted thickness). Lime treatment would therefore be advantageous only where no backfill or improved subgrade material is economically available.

There are also problems in obtaining a uniform and intimate mix. Generally, the lime would have to be added in two or even three increments with intervening 'fallow' periods. Intense pulverising and mixing would also be necessary. Wet weather would make this operation almost impossible due to the extremely sticky character of these soils when wet.

Lime treatment is likely only to be economically viable when pavement savings can be made by taking advantage of the enhanced strength of the treated clay.

6.2.4.5 Minimising Moisture Changes and Consequent Movements

If none of the above methods can be employed, because of an excessive cost or the absence of suitable fill or replacement material, expansive clays may be used for fill and subgrade. Special practices are then necessary to avoid detrimental moisture and volume changes in the swelling soils, as described below.

a. Confining Expansive Clays Under Improved Subgrade and Protective Blankets

As stated in Section 7.2.6, it is in all respects advantageous to place an improved subgrade on native soils of Class S1. Even if a long haul is required, the use of an improved, subgrade will generally prove economical, as it allows a substantial reduction in the sub-base, which is made from much more costly materials.

An improved subgrade has the further advantage of protecting the expansive clays from seasonal moisture changes.

In the case of black cotton soils, it is recommended that the improved subgrade be at least 300 mm thick (compacted thickness). To obtain a good protection against moisture movements, it is desirable that the improved subgrade is impermeable.

Similarly, black cotton soils may be used to form shallow embankments (up to about 3 m), provided that a protective blanket is placed on the slopes, to prevent moisture changes in the black cotton soil. The blanketing material should be at least of S4 quality and be impermeable and resistant to erosion. The blanket thickness should be at least 300 mm.

It is also possible to use black cotton soils to form the cores of higher embankments, the sides being made from good quality fill material. However, due to the complexity of such embankment construction, this practice will rarely be beneficial.

b. Surcharging Expansive Clays

Placing a substantial thickness of non-swelling material over expansive clay reduces heave. The minimum thickness required depends on the expansion pressure of the swelling soil but will usually be 1 – 3 m to produce a useful reduction of swell. It is therefore possible to use black cotton soil to form the lower part of an embankment. It is recommended that the total thickness of pavement plus improved subgrade be at least 600 mm, irrespective of the other protective measures taken.

c. Limiting the Compaction of Expansive Clays

Expansion pressure and potential volume change increase significantly with the dry density of swelling soils. High degrees of compaction may therefore be detrimental and should be avoided.

It is recommended that the dry density of black cotton soils averages around 97 – 98 % MDD (Standard compaction -AASHTO T99) and in no case exceeds (100 % MDD (Standard Compaction).

d. Placing Expansive Clays at Equilibrium Moisture Content

This should prevent moisture changes. If possible, the equilibrium moisture content should be measured under existing roads in the region concerned. Otherwise, it can be assumed that the equilibrium moisture content is near the plastic limit or the Optimum Moisture Content (modified compaction), which is usually close to the plastic limit; this applies in areas where the mean annual rainfall exceeds 500 mm, and the water table is non-existent or deep (more than 5 – 6 m).

In arid areas or in the case of a water table close to ground level, a special study will be required to determine the equilibrium moisture content.

e. **Preventing Moisture Changes Under the Pavement**

It is essential that the swelling soils under the pavement are protected from moisture changes, whether caused by external water or by internal variations (seasonal desiccation - wetting cycles). To this effect, the following practices shall be adopted:

1. The pavement shall be as impermeable as possible. In particular, the use of an impervious bituminous surfacing is required (multiple surface dressing or dense premix).
2. The shoulders shall be impervious and, if necessary, sealed or even surface dressed.

   They shall be extended to a width at least equal to the depth of the zone affected by seasonal moisture changes. Their width shall in no case be less than 2 m.
3. The side ditches shall be dispensed with or, if this is impractical, they shall be located.as far away as feasible from the pavement. They shall have sufficient section and grade to ensure that no water ponding can occur.

   As the areas covered with black cotton soils are generally flat, mitre drains are required at frequent intervals to discharge the water on to the surrounding ground. (See RDM Volume 2 Part 2).
4. Trees shall not be allowed to grow near the road, as their roots may spread under the pavement and produce moisture changes, hence shrinkage and expansion.

6.2.4.6 Type of Pavement Recommended Over Expansive Clays

It is strongly recommended that construction of flexible pavements only should be carried out in the first stage. Indeed, small differential movements of the subgrade are almost inevitable, even if all the above protective measures have been implemented. Flexible pavements can follow these deformations, whereas rigid or semi-rigid pavements would fracture.

Wherever possible, the surfacing of such flexible pavement should be surface dressing which has the further advantage of being impervious. Double surface dressing is sufficient if the pavement layers are impermeable. Triple surface dressing is required if they are permeable.

6.2.4.7 Slopes in Expansive Clays

It is necessary that all fill slopes in black cotton soils be protected by a blanket of non-swelling material. In this case, the slopes indicated in Section 5.3.4 can be adopted.

If unprotected, black cotton soils are prone to erosion and creep due to lateral expansion movements. Safe slopes in black cotton soils are therefore very gentle. If black soils are to be left exposed, as may be the case in shallow cuttings or side ditches, it is recommended that the slope does not exceed 1 (Vertical): 4 (Horizontal).

6.2.4.8 Culverts and Drainage Pipes

Culverts shall not be cast, and drainage pipes shall not be laid, directly against expansive clays. Surrounds or haunches, made from non-swelling material, shall be placed. The use of impermeable and, if possible, gravelly material is essential.

6.3 Dispersive Soils

Dispersive soils are those soils that, when placed in water, have repulsive forces between the clay particles that exceed the attractive forces. This results into the colloidal fraction going into suspension. In moving water, the dispersed particles are carried away or eroded. This obviously has serious implications in earth dam construction but is of less consequence in road construction. However, inclusion of dispersive soils in the subgrade or fill can lead to significant pavement failures through piping, tunnelling, and the formation of cavities.

Dispersive soils often develop in low-lying areas with gently rolling topography and relatively flat slopes. In Kenya dispersive soils occur in Mai Mahiu, Suswa, and Lake Turkana areas.

It is important to identify dispersive soils prior to design. In areas of sloping topography where dispersive soils exist, a characteristic pattern of surface erosion is evidenced by jagged, sinuous ridges and deep rapidly forming channels and tunnels.

In the laboratory, dispersive soils may be identified in the following ways:

1. Crumb test;
2. Pinhole test;
3. CBR test; and,
4. Double hydrometer test.

Crumb Test

The Crumb test is a simple indicator test that is recommended for initial field identification of dispersive soils. The presence of dispersive clays can be identified by observing behaviour of a few crumbs of soils placed in distilled water. Based on the observation of the behaviour of, dispersive soils can be classified as levels 1 to 4 as indicated in Table 6.3 with Levels 1 and 2 representing a non- dispersive reaction, and levels 3 and 4 a dispersive reaction.

Table 6.3 Crumb Test Dispersive Reaction

Level	Observation	Reaction
1	Crumbs slakes or runs out to form a sallow heap at the bottom of the breaker without sign of cloudiness caused by colloidal suspension.	No reaction
2	A very slight cloudiness can be seen in the water at the surface of a crumb.	No reaction
3	There is an easy recognizable cloud of colloids in suspension usually spreading out in thin streaks at the bottom of the beaker.	Moderate reaction
4	A colloidal cloud covers most of the bottom of the beaker, usually a thin skin. In extreme cases all the water becomes cloudy.	Strong reaction

CBR Test

Another early indication of potentially dispersive soils is that the CBR test gives low strengths, less than 3 %, however, this is not a confirmatory test.

Double Hydrometer Test

The Double Hydrometer Test is recommended as indicative laboratory testing for identification of dispersive soils. The test procedure is as follows:

1. Determine the percentage passing 0.005 mm with standard hydrometer testing using a chemical dispersing agent such as ordinary sodium hexametaphosphate; and,
2. Carry out a separate hydrometer analysis using no dispersing agent and determine the percentage passing 0.005 mm.

The percentage dispersion is defined as:

Dispersion = [(% passing 5 μm from Item 2)/(% passing 5 μ from Item 1] x 100 %

Dispersion values greater than 30% are significant and greater than 60% are critical.

Pinhole Test

The pinhole test is a physical test whereby water under various heads of pressure is caused to flow through a hole of 1 mm diameter is a sample specimen. Erosion and widening of the hole is observed and dispersive properties derived from the results.

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From a road engineering perspective, field observation and the first two tests are sufficient guide in deciding whether the soil/subgrade needs special treatment.

The remedial measures applicable on dispersive soils are as follows:

1. Adopt designs that minimise the need for excavation and subsoil exposure, and disturbance to topsoil and vegetation.
2. Cover dispersive soils with a minimum 100 mm layer of non-dispersive soil prior to re-vegetation or placement of the pavement layers.
3. Avoid its use in fills as much as possible.
4. Remove and replace it in the upper 300 mm of the subgrade.
5. Top dress the surface of potentially dispersive soils with up to 2 % gypsum if soil pH > 6.5 or up to 4 % lime if soil pH < 5.0 or a mixture of both if soil pH is between 5.0 and 6.5. The use of gypsum is recommended over lime, as lime may lead to soil stabilisation and its associated cracking.
6. Manage water flows and drainage in the area well.
7. Infill any trenches or holes to prevent collection and ponding of water on subsoil surfaces.

6.4 Collapsible Soils

Collapsible soils are those that appear to be strong and stable in their natural dry state, but which rapidly consolidate under wetting, generating large and often unexpected settlements. Such soils include loose and windblown silts generally consisting of 50 to 90 % silt particles.

Often, the loose structure of these soils is held together by small amounts of clay minerals or calcium carbonate. The introduction of water dissolves the bonds created by these cementing materials and allows the soil to take a denser packing under any type of compressive loading. The condition for collapse is that the soil mass must be in a partially saturated condition and then wetted up and loaded simultaneously which can occur beneath pavement structures.

Collapsible soils possess porous textures with high void ratios and relatively low densities. They often have sufficient void space in their natural states to hold their liquid moisture at saturation.

In the laboratory the potential degree of collapse is best determined using the oedometer test. Information on the general location of collapsible soils in Kenya should be obtained from the Chief Engineer (Materials). However, confirmatory investigation should be carried out where the project road traverses such soils.

The oedometer test can be used to assess the degree of collapsibility. The double oedometer test for assessing the response of a soil to wetting and loading at different stress levels should be carried out.

The Collapse Potential (CP) is defined as (Equation 6.1):

$$CP\ (\%) = \frac{\Delta e_c}{1 + e_0} \times 100$$

Equation 6.1

Where,

Δe_c = the change in void ratio at 200 kPa, upon wetting

e_0 = the initial void ratio

Since the test is conducted as one-dimensional test the collapse potential can also be expressed as (Equation 6.2):

$$CP\ (\%) = \frac{\Delta h}{h_0} \times 100$$

Equation 6.2

Where,

Δh = the change in height upon wetting

h_0 = the initial height of the specimen

This test is a guide or indicator test only to determine the potential for collapse (see Table 6.4) settlement to occur and is merely a qualitative indication.

Table 6.4 Features of Collapsible Soils

Collapse Potential (%)	Severity
0 - 1	Very low
1 - 5	Low
5 - 10	Moderate
10 - 20	High
> 20	Very high

Remedial measures include:

1. Moistening and using high energy compaction techniques using large impact rollers to reduce the collapse potential.
2. Ground modification involving either partial removal and replacement or densification of the collapsible soil using techniques such as compaction grouting and pre-wetting of the soil followed by a surcharge loading to cause settlement before construction. These methods include partial removal and replacement; densification of the collapsible soil in-place such as by compaction grouting; and pre-wetting of the collapsible soil followed by surcharge loading to cause settlement before construction; conventional compaction with heavy vibratory roller for shallow depths (within 0.3 to 0.6 m); and dynamic or vibratory compaction for deeper compressible soils of more than 0.5 m (combined with inundation with water). For collapsible soils with thickness greater than 1.5 m, lime pressure and sodium silicate injections could also be helpful, though expensive.

6.5 Saline Soils

The presence of soluble salts, i.e. NaCl, Na2CO3, NaHCO3, (but not gypsum, Na2SO4, which is only slightly soluble) in pavement or earthwork materials, or more critically in the subgrade and/or groundwater can cause damage to prime coats and thin surfacings.

This is a significant risk in arid climates because of the migration of these salts to the surface as a result of evaporation. Coastal areas, or possibly areas in the vicinity of the saline lakes in Kenya, are most at risk from this mode of damage.

The content of soluble salt can be rapidly but indirectly determined by laboratory or field determination of electrical conductivity. It is prudent to determine the precise configuration of soluble salts by chemical analysis on a few samples and relate this to the conductivity. If the tests are carried out on potential pavement materials, rather than subgrade, construction water should be added at 1.5 times the required amount to obtain the OMC, to allow for evaporation, before the sample is tested.

The total soluble salts (TSS) will normally be measured; however, it is important to ascertain the dominant salt type for more detailed design and construction control, particularly if salt levels are significant. Methods have been developed whereby the electrical conductivity (EC) of an aqueous solution of the material is measured using a standardized procedure and the values are related to the percentage of TSS of the sample (Equation 6.3).

$$TSS = 0.04 + 0.16EC$$

Equation 6.3

Where,

EC = measured in millisiemens per centimeter siemen is the reciprocal of the electrical resistance ohms at 25 °C, and the TSS content is measured in mass percentage.

Prime coats are very vulnerable to the formation of blisters in the bituminous surfacing and by fretting of the edges of the surfacing. If the soluble salt content, measured as % Total Soluble Salt (TSS), exceeds approximately 0.3 % in the upper 50 mm of the road base, they are susceptible to damage. Cutback prime is more vulnerable than emulsion prime. Blistering damage is accelerated if the road is low-trafficked.

Surface dressing is more resistant to attack. Single and double surface dressings are not susceptible to damage unless the % TSS exceeds 1.0 %; however, if surface dressings are constructed on saline subgrades, it is recommended that an impermeable fabric be placed beneath the road base to prevent the upward rise of salt and protect the surface dressing from eventual salt damage. If the road is well trafficked, the susceptibility to damage is reduced.

6.6 Organic Soils

These commonly occur in swamp areas and require special investigations to evaluate ground stability and potential for excessive settlement. Typically, remediation consists of surcharging the pavement structure for a specified time before removing the surcharge and constructing the pavement. However, secondary consolidation of organic soils can be significant, and caution should be exercised when specifying pavement construction on top of an embankment when 90 % of primary consolidation (based on the consolidation test results) has taken place. Other remediation measures comprise removal and replacement of the organic soil or, in extreme cases, construction of the road floating on the swamp material.

A high content of organic matter (>2 %) is undesirable in pavement materials, particularly in stabilised layers because it causes increased demand for stabiliser to achieve the required strength.

6.7 Halloysite

The presence of halloysite as a major constituent of the residual soils accounts for or contributes to the following properties:

- The position of test results to the right of the 'A' line on the plasticity chart.
- Higher than normal moisture contents, for example for the compaction optimum.
- High strength in terms of effective stress parameters.

While these properties are unusual, halloysitic soils have been used in the construction of a number of major dams with no deleterious effects. In other terms its presence in road embankments should not be a problem.

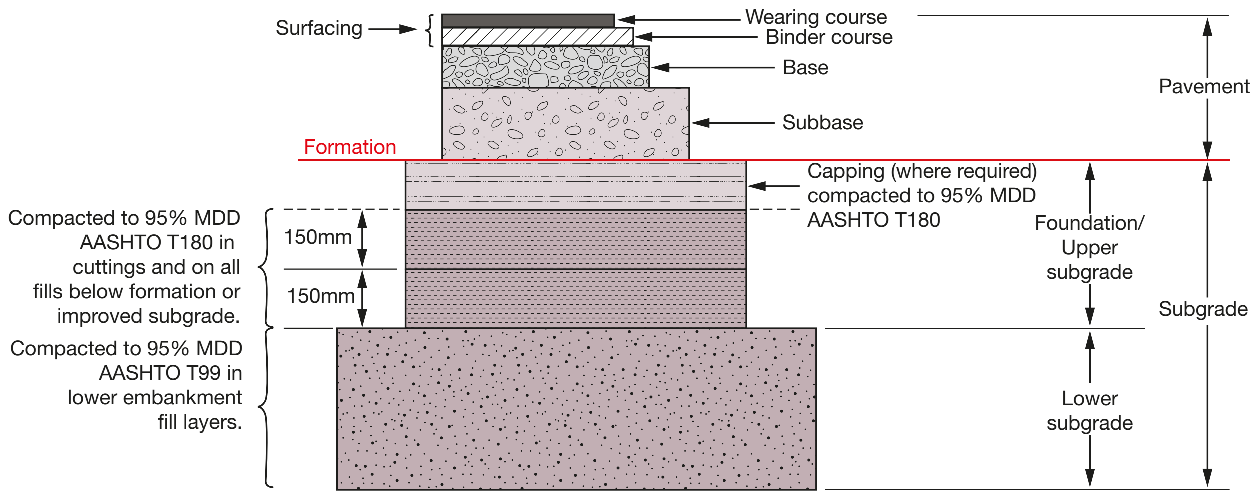
Besides, the principal influence of halloysite appears to be that the engineering properties of the soil are good, despite a high clay fraction and very small particle size, and high values of natural water content and Atterberg limits. The good engineering properties appear to be the direct result of their mineralogical composition, or in some cases cementation arising from the presence of the sesquioxides.

7 Pavement Foundations

7.1 General

The foundation is the upper subgrade below the formation. Where no capping (improved subgrade) is provided, it is the upper 300 mm of the subgrade below the formation. Where a capping layer is provided, the foundation shall be defined as the combination of the top 300 mm of fill or cutting and all the capping layers provided. The sub-base is not part of the foundation, it is considered a pavement layer. The illustration is shown in Figure 7.1.

Figure 7.1 The Foundation and the Pavement



7.2 Subgrade Requirements for Pavement Design

7.2.1 Subgrade Compaction

The compaction requirements shall be generally as follows:

1. The upper 300 mm of the subgrade shall be compacted to a dry density of at least 95 % MDD AASHTO T180 (Heavy compaction) in cuttings and on all fills below formation or improved subgrade.
2. All improved subgrades shall be compacted to a dry density of at least 95 % MDD AASHTO T180 (Heavy compaction).
3. The maximum compacted thickness processed and compacted at one time shall be:
   i. 150 mm for the upper 300 mm of the subgrade in cuttings and fills; and,
   ii. 150 mm unless through a trial, it is demonstrated that a 200 mm layer can be compacted at one time.

The lower embankment fill layers below the upper 300 mm of the subgrade shall be compacted to a dry density of 100 % MDD AASHTO T99 (Standard compaction).

The compaction requirements are summarised in Table 7.1.

Table 7.1 Compaction of Subgrade

Subgrade Layer	Description	Compaction Specification
Upper subgrade (Pavement foundation)	The top 300 mm of the subgrade below formation level in cuttings and fills where there is no improved subgrade or the upper 300 mm of the subgrade in cuttings and fills and improved subgrade.	• 95% MDD (AASHTO T180) on fills, cuttings, and improved subgrade layers • The maximum compacted layer at any one time shall be 150 mm on fills and cuttings and 200 mm on improved subgrade layers.
Lower subgrade (Cuts or fill embankments)	Cuts or fill embankments below the foundation/upper subgrade	• 100% MDD (AASHTO T99) on embankment fills • The maximum compacted layer thickness shall be 150 mm, unless as a result of trials it is demonstrated that compaction can be consistently achieved at greater depth which should not exceed 250 mm.

The moisture content shall be adjusted in order that the required relative compaction is obtained, but the moisture content at the time of compaction shall not exceed 105 % of the Optimum Moisture Content (the specified compaction level).

The moisture content shall be measured using the Nuclear Density Gauge, another moisture measuring device that gives speedy results (e.g. calcium carbide devices), or by taking a sample and oven-drying.

In some cases, it is advantageous to obtain relative compactions higher than the above figures, since compaction not only improves the subgrade bearing strength, but also reduces permeability. This particularly applies to clayey sands, silty sands and granular materials, the coarse particles of which are hard enough not to crumble under heavy compaction.

7.2.2 Estimating the Subgrade Moisture Content

The actual moisture content of the subgrade soil under the road pavement will depend on many factors, principally:

1. Local climate (Figure 7.2) shows the Koppen-Geiger climate classification of Kenya and shall be used in determining the climatic zone of road projects).
2. Depth of the water table.
3. Type of soil.
4. Topography and drainage.
5. Permeability of the pavement materials.
6. Permeability of the shoulders.

After the pavement is constructed, the moisture content of the subgrade will generally change. In warm desert and warm semi-arid climates, the moisture content is expected to decrease. In areas where the water table is present, the subgrade moisture content may or may not change subject to the depth of the water table. Despite this, the moisture content used for subgrade strength assessment shall be determined after 4 days of soaking. (Please refer to Figure 3.2 in this Manual).

7.2.3 Determining the Subgrade Design Strength

Unless a more accurate estimation of the ultimate subgrade moisture content can be made and backed by factual data, the subgrade strength shall be determined based on CBRs measured after 4 days soaking for all climatic classifications.

7.2.4 Subgrade Failure Criterion

It is widely accepted that the compressive strain on the surface of the subgrade is the criterion that governs the total cover required in the case of a flexible pavement. If the compressive strain is excessive, permanent deformation will occur in the subgrade and this will cause deformation at the pavement surface.

The relationship between the maximum permissible compressive strain and the cumulative number of standard axles is usually given by an empirical equation called the 'subgrade failure criterion', which relates the vertical strain at the subgrade level to pavement rutting performance.

The general relationship is as described in Equation 7-1.

$$N = \left(\frac{A}{\varepsilon}\right)^n$$

Equation 7.1

Where,

ε = In micro-strain (i.e. strain/1,000,000).

N = The number of cumulative equivalent standard axles.

A and n = Constants.

In practice this relationship depends on the strength of the subgrade, hence there are a variety of relationships in international literature. Those most relevant are given in Table 7.2 and shown in Figure 7.2.

Table 7.2 Subgrade Strain Criteria

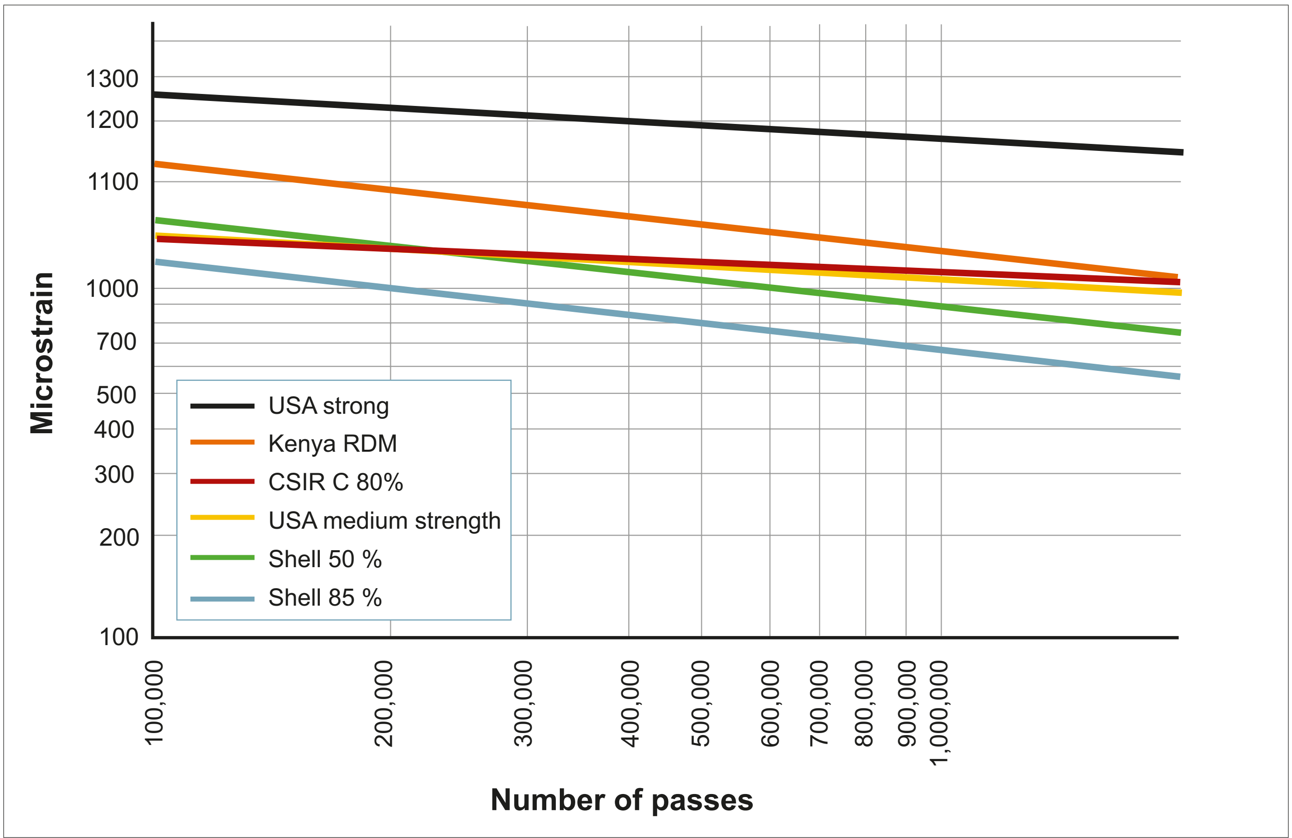
Constant	Subgrade Strain Criterion					
	Shell 50 %	Shell 85 %	CSIR C 80%	USA Strong	USA Medium	Kenya RDM III
A	28,000	21,000	36.47[1]	19,700	10,900	41445
n	4	4	-	5.714	5.714	4

***Note 1:** N = 10^(A-10*logε), Where ε is the vertical strain on top of the subgrade*

In Kenya, the Shell 50 % criterion is applicable to subgrades for design traffic greater than 1 MCESA, and the Kenyan criterion is applicable to subgrades for design traffic less than 1 MCESA (LVSRs).

The Shell 85 % criterion is applicable to very weak subgrades (<3 % CBR), the USA medium strength and the CSIR C criteria are applicable to subgrades of CBR greater than 5 % but less than 8 %, and the Kenyan criterion is applicable to slightly stronger subgrades. Finally, the USA strong criterion is applicable to the subgrade of CBR > 20 %, but these ranges should be considered as very approximate.

Figure 7.2 Permissible Vertical Strain on the Subgrade



7.2.5 Subgrade Stiffness Modulus

The design system incorporates the dynamic elastic modulus of the subgrade as one of the principal design parameters.

The modulus should correspond to the common moisture content that the subgrade soil will endure under the pavement, since the effects of repeated loading are considered.

The moduli of the subgrade classes at their equilibrium moisture contents have been determined by direct measurements (plate bearing tests), and the values are given in Table 7.3.

Table 7.3 Subgrade Surface Moduli

Subgrade Class	CBR at 100 % MDD (AASHTO T99) and 4-day soak (%)		Surface Modulus (MPa)
	Range	Median	
S1	2 – 5	3.5	40
S2	5 – 10	7.5	65
S3	7 – 13	10	75
S4	10 – 18	14	95
S5	15 – 30	22.5	130
S6	30 – 60	45	200

7.2.6 Empirical Relationships for CBR to the Subgrade Resilient Modulus

The Resilient Modulus (MR) of soil is a measure of subgrade stiffness. It can be obtained using the cyclic triaxial test. The determination of MR in the laboratory is time-consuming and hence empirical relationships have been developed.

Equation 7.2 derived by Powell et al (1984), shall be used to estimate the short-term and long-term subgrade service modulus.

$$M_R = 17.6 * CBR^{0.64}$$

Equation 7.2

Where,

M_R = the estimated subgrade surface modulus in MPa; and,

CBR = the California bearing ratio of the subgrade in %.

Equation 7.2 is valid for CBR values in the range of 2 to 12 %. However, research at MTRD shows that it is valid up to CBR 100 %. Above CBR 12 %, direct measurement in the laboratory or in-situ is advised.

The equivalent subgrade modulus for each of the subgrade classes is given in section 7.2.5.

7.3 Foundation Classes

Pavement design is carried out in two stages, foundation design and pavement structure design. The foundation carries the construction traffic and acts as a platform for the construction of the upper pavement structure. The long-term properties of the foundation are used in analytical models to design the pavement structure.

The design thicknesses are based on foundation classes defined in terms of the subgrade surface modulus at equilibrium moisture. The foundation classes selected for design and their applicability are presented in Table 7.4.

Table 7.4 Pavement Foundation Classes and Applicability

Foundation Class	Minimum Foundation Requirement		Equivalent Subgrade Class	Applicable Design Traffic	
	Surface Modulus (MPa)	Equivalent Minimum CBR (%)		Traffic Load Category	Traffic Class
F1	75	10	S3	Low	TC0.025 -TC1
F1	75	10	S3	Medium	TC3 – TC10
F2	95	14	S4	Heavy	TC17 – TC30
F3	130	23	S5	Heavy	TC50
F4	200	45	S6	Very Heavy	TC80 and TC150
F5	400	140	N/A		TC80 and TC150+

7.4 Improved Subgrade

7.4.1. General

Placing an improved subgrade not only increases the bearing strength of the direct support of the pavement, but also:

1. Protects the upper layers of earthworks against adverse weather conditions and their effects such as soaking and shrinkage.
2. Facilitates the movement of construction traffic.
3. Permits proper compaction of the pavement layers.
4. Reduces the variation in the subgrade bearing strength, and
5. Prevents pollution of open-textured sub-bases by plastic fines from the natural subgrade.

It may prove technically and economically advantageous to lay an improved subgrade not only on S1, but also on other subgrade classes (S2 to S6). The decision will generally depend on the respective costs of sub-base and improved subgrade materials, and on the design traffic class as presented in Table 7.4. Improvements to achieve higher foundation classes, F4 and F5, are required for rigid pavements

An improved subgrade placed on soils must be made of a material of a higher class as discussed in section 7.4.3.

7.4.2 Fill

The best materials either from cuttings or from borrow areas, should be reserved for the upper layers of fill. Fill in soft material shall be selected based on determined in-situ subgrade class, i.e., on a section traversing Class S2 subgrade soils, material of CBR of 5% to 10% shall be used as fill material and the top 300 mm using selected material of CBR ≥ the class median CBR strength (G8). This applies for the other subgrade soils except for class S1 (CBR 2-5%) for which the minimum requirements are as specified for G3 in Table 7.5. Materials for fill shall meet the requirements in Table 7.5.

7.4.3 Capping Layers (Improved Subgrade)

Where a sufficient thickness of improved subgrade is placed, the overall subgrade bearing strength is increased to that of a higher class and the sub-base thickness may be reduced accordingly. There may also be problems in achieving high degrees of compaction in Class S4 or S5 material overlying a Class S1 soil. The minimum thickness of each class of improved fill required on each class of natural subgrade soil to obtain a higher class of subgrade bearing strength is shown in Table 7.6.

If the material in cuttings or embankments does not meet the requirements of the selected foundation class, a capping layer shall be constructed to improve the subgrade to achieve the design foundation class.

The improvement is designed to bring the existing native subgrade plus the capping layers up to an overall bearing strength level equivalent to that of the selected foundation.

Table 7.5 Requirements for Fill and Improved Subgrade

Determined Native/ In-situ Subgrade Class	Requirements for Fill in Soft Material		
	Lower subgrade (Fill below the pavement foundation[1])	Upper subgrade (Pavement Foundation)	
		Top 300 mm of the **subgrade** below the formation	Top 300 mm of **fill** below improved subgrade[2]
Below Class S1: CBR < 2%	**CBR ≥ 3%** **Swell:** < 3% **LL:** ≤ 70% **PI:** ≤ 50%	N/A	G3
S1: CBR 2-5%	**CBR ≥ 3%** **Swell:** < 3% **LL:** ≤ 70% **PI:** ≤ 50%	N/A	G3
S2: CBR 5-10%	**CBR ≥ 5%** **Swell:** < 3% **LL:** ≤ 70% **PI:** ≤ 50%	N/A	G8
S3: CBR 7-13%	**CBR ≥ 7%** **Swell:** < 2.5% **LL:** ≤ 50% **PI:** ≤ 35%	G10 for foundation Class F1	G10
S4: CBR 10-18%	**CBR ≥ 10%** **Swell:** < 2.5% **LL:** ≤ 50% **PI:** ≤ 30%	G14 for foundation Class F2	G14
S5: CBR 15-30%	**CBR≥ 15%** **Swell:** < 2% **LL:** ≤ 35% **PI:** ≤ 25%	G23 for foundation Class F3	G23
S6: CBR 30-60%	**CBR > 30%** **Swell:** < 0.5% **PI:** ≤ 12%	G45 for foundation Class F4	G45

__1.__ The foundation is the upper subgrade below the formation. Where no capping (improved subgrade) is provided, it is the upper 300 mm of the subgrade below the formation. Where a capping layer is provided, the foundation shall be defined as the combination of the top 300 mm of fill or cutting and all the capping layers provided. The sub-base is not part of the foundation, it is considered a pavement layer.

__2.__ Material specifications for G3, G8, G10, G14, G23, G30 and G45 are provided in Table 8 8 and in Material Charts GM1, GM2, GM3, GM4, GM6, and GM9, respectively.

The minimum thickness of each type of capping material required to improve the subgrade to a higher class is shown in Table 7.6. The minimum thicknesses have been calculated considering the respective elastic modulus of each class of soil. It should be noted that the foundation layers are additive. For example, to improve a S2 subgrade to a S6 (F4) equivalent, it can first be improved to an S4 (F2) by adding 175 mm of G14, followed by 150 mm of G45 into an S6 (F4). The specifications for the capping materials in Table 7.6 are found in section 8.9.

In special circumstances, such as due to scarcity of natural materials, higher quality materials such as crushed rock and hydraulically modified or bound materials may be used as capping. Due to the high cost this would present, their use must be fully justified by the designer.

Table 7.6 Minimum Capping Thicknesses for Improved Subgrade

Native Subgrade Class	Improved Subgrade/Capping		New Subgrade Class	Foundation Class
	Material	Minimum Thickness (mm)		
S1	G8	375	S2	N/A
	G10	300	S2	N/A
	G10	400	S3	F1
	G14	250	S2	N/A
	G14	350	S3	F1
	G14	425	S4	F2
S2	G10	150	S3	F1
	G14	150	S3	F1
	G14	175	S4	F2
S3	G14	150	S4	F2
	G23	150	S4	F2
	G45	150	S5	F3
	G45	250	S6	F4
	HIG100	250	S6	F4
	BSM50	275	S6	F4
S4	G23	150	S5	F3
	G45	200	S6	F4
	HIG100	200	S6	F4
	BSM50	225	S6	F4
S5	G45	150	S6	F4
	GCS/BSM50	275	S6	F4
	HIG160/HMS1	125	S6	F4
	HIG160	250	N/A	F5
	HMS1	225	N/A	F5
S6	GCS	250	N/A	F5
	HIG160	200	N/A	F5
	HMS1	175	N/A	F5
	BSM100	150	N/A	F5

For concrete pavements, only F4 and F5 foundations should be used, and the top-most capping layer must be HIG160, HMS1, BSM100 or higher quality BOUND material. To achieve this for S1 and S2 subgrades, they must first be improved to S3 (F1) or S4 (F2) subgrade class before final improvement to F4 and F5.

7.5 Lime and Cement-treated Subgrade

Treatment of the subgrade soils with lime, cement, other binder, may be considered in the following cases:

1. Where the natural soils are excessively clayey and no better material is economically available, their treatment with hydrated lime may be the cheapest solution.
2. Where the natural soils are excessively wet and cannot be dried out because of adverse weather conditions, their treatment with quicklime may allow construction to proceed and provide a markedly stronger subgrade.

Lime and cement-treatment shall be considered for subgrade soils with the material properties as presented in Table 7.7.

Table 7.7 Material Properties for Stabilisation Treatment With Lime and Cement

Stabiliser	Soils with more than 35% particles passing the 0.075 mm sieve			Soils with less than 35% particles passing the 0.075 mm sieve		
	PI ≤ 10	10 < PI < 20	PI ≥ 20	PI ≤ 6, PP ≤ 60	PI ≤ 10	PI ≤ 10
Lime	Marginally effective	Yes	Yes	Not suitable	Marginally effective	Yes
Cement	Yes	Yes	Marginally effective	Yes	Yes	Yes

When assessing the CBR of lime or cement-treated material, consideration should be given to measuring the CBR and swell of the untreated material to provide an indication of the effectiveness of the lime treatment. The treated soils will be classified as improved subgrade and achieve at least twice the CBR strength of the improved subgrade material specified in Table 7-6, i.e., for G14 material the treated material should achieve a minimum CBR of 28% after 7 days soak.

7.6 Performance Foundations

7.6.1 General

Pavement foundations are classified as restricted/empirical foundation, or as performance foundation. Restricted foundations are those that have been developed through empirical studies or from past performance experience. In this manual these are generally classes F1 to F3. Performance foundations are those that are designed through practical experimentation and testing on site during the works execution. In this manual these are generally classes F4 and F5.

7.6.2 Minimum Deflection Requirements

These are applicable for design traffic class TC 80 or higher. The following shall be detailed for each foundation design:

1. Start and end chainage.
2. Foundation class.
3. The layer stiffness.
4. Estimated minimum layer thicknesses based on Table 7.6 or structural analysis

Performance foundation design thicknesses shall be derived using constructed trial sections with various materials options. After construction of the trial sections in accordance with standard specifications, the layers/sections shall be wetted for 4 continuous days to simulate soaked conditions. The sections shall then be tested and in case of failure, layer thicknesses shall be increased (topped up) or different materials trialled.

Note that protection of the subgrade during construction (short-term) is based on the vertical compressive strain in the top of the subgrade. The structural response is limited so that excessive deformation does not occur. The deflection under a given load can be equated to a surface modulus for the foundation as a whole.

The deflection of the foundation shall be measured using an FWD (40 kN load over a 150 mm radius loaded area) or calibrated LWD and the maximum deflection of the foundation for each foundation class shall be no greater than:

1. 1.65 mm (1650 μm) for Foundation Class 1 (75 MPa).
2. 1.34 mm (1340 μm) for Foundation Class 2 (95 MPa).
3. 0.95 mm (950 μm) for Foundation Class 3 (130 MPa).
4. 0.74 mm (740 μm) for Foundation Class 4 (200 MPa).
5. 0.37 mm (370 μm) for Foundation Class 5 (400 MPa).

The surface modulus at the top of the foundation is calculated as in Equation 7.3 below:

$$E_o = 2(1 - \upsilon^2)\sigma_o \frac{a}{\delta_o}$$

Equation 7.3

Where.

E_o = the surface modulus at the centre of the loading plate in MPa,

υ = the Poisson’s ratio taken as 0.35,

σ_o = the contact pressure under the loading plate in kPa,

a = the radius of the loading plate in mm,

δ_o = the deflection at the centre of the loading plate in μm.

The Poisson's Ratio value used for the design of capping materials shall be 0.35.

7.6.3 Procedure for Field Determination

7.6.3.1 General

Performance foundation designs shall be subject to performance testing. Capping materials for construction of F4 and F5 foundation shall be as follows:

1. **Class F4 foundation:** Gravel of CBR ≥45% (G45), graded crushes stone class E (GCS-E), and hydraulically improved materials of CBR ≥ 100% (HIG-100); and,
2. **Class F5 foundation:** graded crushes stone class A, or B (GCS-A or B, hydraulically improved materials of minimum CBR of 160% (HIG160, and hydraulically modified stone of UCS ≥ 1 MPa (HMS1).

7.6.3.2 Demonstration Area

A demonstration area shall be constructed and tested to confirm the performance of each foundation design. The demonstration area shall be prepared using the same methods, materials, thickness, and compaction as proposed for the permanent works. Each demonstration area shall be not less than 400 m² and not less than 60 m long and shall have a sufficient and suitable run off/run on area to ensure the trafficking trial can be undertaken.

The materials placed in the demonstration area may form part of the permanent works, provided that they meet the requirements of the permanent works.

7.6.3.3 Determination of the Subgrade Surface Modulus

The subgrade surface modulus shall be determined using one of the following devices:

a. Falling Weight Deflectometer (FWD).

b. Lightweight Deflectometer (LWD).

c. Plate Loading Test.

Detailed procedures for using these devices are included in RDM Volume 5, Part 1.

a. Falling Weight Deflectometer (FWD) Testing

The FWD is the preferred and first choice tool for the design of performance foundations. FWD testing shall be undertaken using a calibrated FWD in accordance with BS 1924-2 (2018 section 9.3).

b. Lightweight Deflectometer Testing (LWD)

In lieu of an FWD, an LWD may be used. LWD testing shall be undertaken using a calibrated LWD in accordance with BS 1924-2 (2018 section 9.2). In accordance with BS 1924-2 (2018 section 9.2), an LWD device shall only be used with a site-specific correlation versus a FWD or if it has an annual correlation certificate.

Testing shall either be undertaken to:

i. Procedure A – the standard target stress as per BS 1924-2; or

ii. Procedure B – a range of target stresses centred around 100 KPa to determine stress dependency.

c. Plate Loading Test

In lieu of an LWD, a plate loading test may be conducted through application of a load to circular plate, and measurement of the result deflection until the point of required design load. Equation 7.4 shall be used for computation of the stress.

$$\sigma_z = \frac{P}{A}\left(1 + \frac{z}{z^2 + a^2}\right)$$

Equation 7.4

Where,

σ_z is the vertical stress at depth z.

P is the applied load.

A is the loaded area.

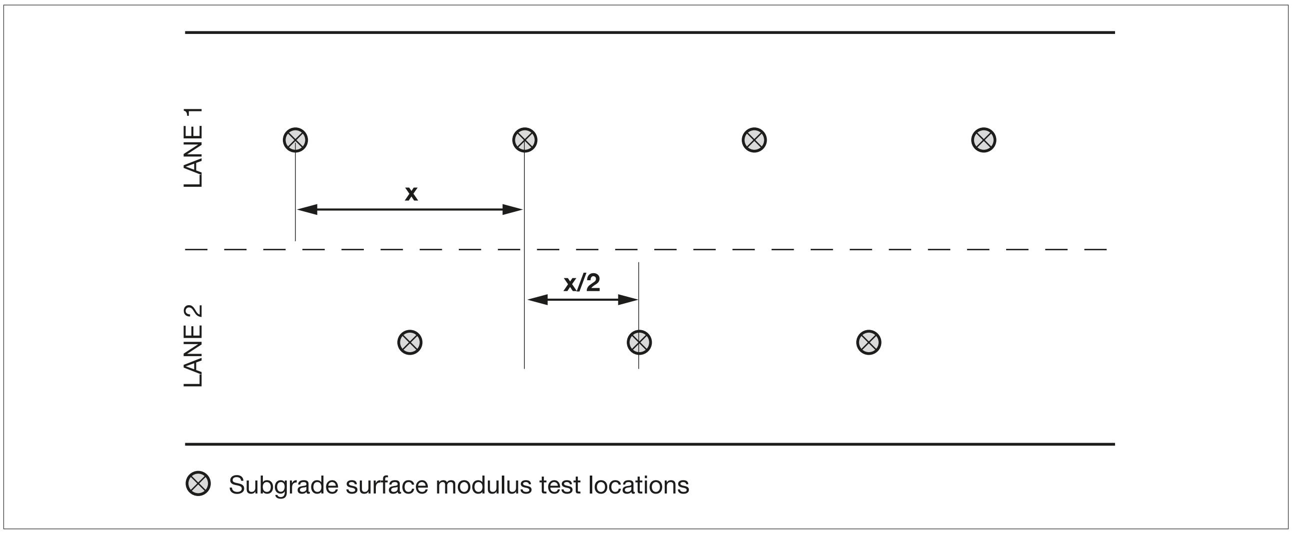
a is the radius of the loading plate.

The process for the performance testing of the demonstration area is as follows:

1. Measure subgrade surface modulus as set out below and confirm that the value is equal to or higher than the design subgrade surface modulus value. If the measured subgrade surface modulus is lower than the design subgrade modulus, a foundation redesign is required.

i. The construction subgrade surface modulus shall be determined at intervals suitable for the type of subgrade material and its condition, with a maximum spacing of 60 m along each lane of prepared subgrade and staggered to the mid-point between adjacent lanes, see Figure 7.4.

Figure 7.3 Subgrade Surface Modulus Test Locations



- **ii.** At least 10 tests shall be carried out for each prepared foundation area.
- **iii.** The measurement of the construction subgrade surface modulus shall be taken at formation level or at sub-formation level if capping is part of the foundation design.
- **iv.** The test site shall be free from standing water.
- **v.** Where the construction subgrade surface modulus is found to be less than the design subgrade modulus, the area shall either be improved, or the foundation redesigned.

2. Where the measured subgrade surface modulus is greater than design subgrade surface modulus, an adjustment factor should be applied to the measurements of foundation surface modulus values using Equation 7.5 and used for the pavement design.

$$E_F\,(adjusted) = E_F\left[1 + \left(0.28 * Ln\left(\frac{E_S\,(measured)}{E_S\,(measured)}\right)\right)\right]$$

Equation 7.5

Where,

$E_F\,(adjusted)$ = The adjusted foundation surface modulus.
E_F = The measured foundation surface modulus.
$E_S\,(measured)$ = The measured subgrade surface modulus.
$E_S\,(designed)$ = The design subgrade surface modulus.

3. Construct the foundation as proposed for the permanent works.

4. The Contractor shall undertake a controlled trafficking trial on top of the constructed foundation in the demonstration area.

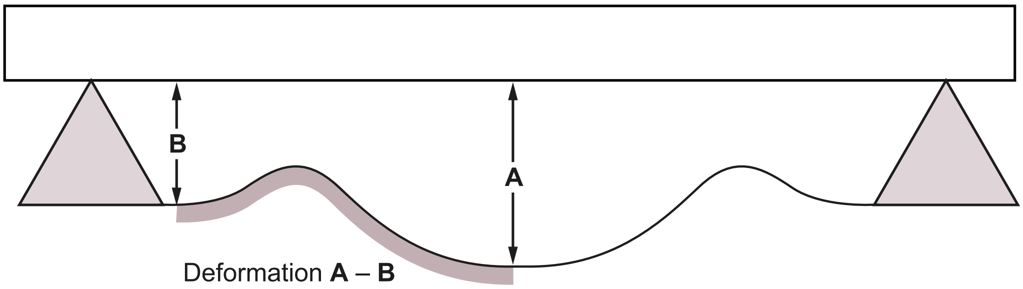
5. The trial procedure shall be as follows:

- **i.** Mark out the intended running track for the vehicle to ensure same wheel path is followed in each pass.
- **ii.** Using a heavy goods vehicle, undertake several passes of the demonstration area along the marked out running track to achieve a number of passes equivalent to 1000 standard axles.

iii. At 10 metre intervals or as instructed by the client, measure vertical deformation in the wheel path using a straight edge with a length of at least 2 m.

iv. The straight edge shall be placed transverse to the rut and raised clear from the rut by two identical blocks. The blocks shall be placed on undisturbed material outside the wheel path. The amount of deformation shall be the difference of the foundation (A) and the height of the blocks (B), see Figure 7.4.

Figure 7.4 Wheel Path Deformation



6. The wheel path deformation shall not exceed the following limits along any section of trial length:
   - All bound surfaces – 10 mm.
   - < 250 mm design thickness of granular material – 30 mm.
   - ≥ 250 mm design thickness of granular material – 40 mm.
7. Repeat measurement of foundation surface modulus to confirm the foundation surface modulus requirements for the foundation are in accordance with the requirements above.
8. Where the completed demonstration area meets all of the specification requirements, the methods, materials and thicknesses used shall not be changed for the construction of the main works without an additional demonstration area being constructed.
9. Where the demonstration area fails to meet all of the specification requirements, the design and construction practices shall be reviewed, and the demonstration area reconstructed until the specification requirements have been met.
10. Records of the performance test results for each construction stage, referenced to the following condition details shall be stored and presented on request to the client in a digital spreadsheet format:

i. Subgrade surface modulus value immediately before foundation construction.

ii. Date and time of mixing (for stabilised and slow-setting materials).

iii. Date and time of placing and compaction.

iv. Date of performance testing.

v. Values of surface modulus recorded.

vi. Values of material properties including density and layer thickness.

vii. Weather conditions including temperature.

viii. Sampling and testing records in the demonstration area.

7.6.3.4 Permanent Works

For the main works, a testing regime shall be detailed to confirm that the foundation surface modulus shall be equal to or higher than that specified for the designed foundation class.

It should be noted that the foundation surface modulus measured is for a partially confined foundation and is not to be confused with the long-term confined foundation surface modulus.

Where the foundation surface modulus is lower than that specified for the designed foundation class, action shall be taken to either undertake improvement or review the foundation design. The approach for improvement is dependent on the scale of the issue and the practical options available on site.

Where a foundation area within the main works fails to comply with the surface modulus performance measurement requirements, and the foundation is to be redesigned, the suitability of the redesigned foundation shall be confirmed with a demonstration area.

For each foundation area, records of the performance test results for each construction stage, referenced to the following condition details shall be stored and presented on request to the Client in a digital spreadsheet format:

1. Subgrade surface modulus value immediately before foundation construction.
2. Date and time of mixing (for stabilised and slow-setting materials).
3. Date and time of placing and compaction.
4. Date of performance testing.
5. Values of foundation surface modulus recorded.
6. Values of material properties including density and layer thickness.
7. Weather conditions including temperature; and
8. Sampling and testing records.

Within the 48 hours prior to construction of the overlying pavement layers, the foundation surface modulus shall be tested at 20 m intervals along each lane, staggered by 10 m between adjacent lanes. Tests shall coincide with subgrade surface modulus and density tests where appropriate. The foundation surface modulus values achieved shall meet or exceed that for the corresponding material and foundation class.

A foundation containing unbound materials that fails to comply with the performance requirements when the recorded moisture content exceeds that of the demonstration area, may be re-tested for compliance when the foundation moisture content has reduced to that in the demonstration area.

Where surface modulus performance values do not meet the requirements of the foundation required, a re-design should be completed.

Ruts that develop under construction traffic, measured in accordance with this Clause, shall nowhere exceed the limits in (7) above.

7.6.4 Thickness Design Requirements

The design thickness shall be derived using the layer stiffness values assigned to each layer and the procedure outlined in Section 7.6.3.

When the subgrade surface modulus is expected to be low at the time of construction, a capping layer should be added to provide a working platform for construction of the subsequent layers. It is noted that foundations built on a construction subgrade surface modulus of 50 MPa or less have a relatively high risk of structural rutting during construction if the foundation does not incorporate a capping layer.

The design thickness derived shall either be subject to zero negative tolerance; or, to ensure that the design thickness is applied throughout the scheme, an additional thickness may be applied to the derived thickness. Any additional thickness required may be limited by the capacity of the construction equipment to deliver the required design thickness consistently.

The minimum capping thickness for foundation class F5 shall be at least 150 mm regardless of the material used.

7.6.5 Use of Geosynthetics

Geosynthetics are categorised as proprietary products. For all proprietary products, performance design is required. This involves the construction of a trial section and conducting several tests to ensure performance compliance.

In such special cases, the design engineer should follow the design and construction principles set out in this Manual and liaise closely with the MTRD for guidance on approval procedure of such materials.

Geosynthetics refer to synthetic materials used in civil engineering and geotechnical applications to enhance the performance of soil and rock. In road construction, four main types of geosynthetics are used.

1. Geogrid.
2. Geotextile.
3. Geomembrane.
4. Geocell.

Geogrids are high-strength synthetic grids or mesh structures made from materials like polyethylene or polypropylene. They are employed for soil reinforcement and slope stabilisation. Geogrids are especially useful in applications where increased tensile strength is required to withstand heavy loads and prevent soil movement.

Geotextiles are permeable fabrics made from synthetic materials such as polypropylene or polyester. They are commonly used to separate different soil layers, prevent soil erosion, and facilitate drainage. Filter fabrics can be categorised according to the method of manufacture as follows:

- Woven geotextiles may be made from single fibres, slit film or bundles of fibres woven together. The resulting geotextiles generally have high tensile strength, high modulus, and low elongation. Woven filter fabrics generally are available in the range of equivalent opening size (EOS) values between 150 and 600 μm. However, the EOS may decrease markedly when a slit film woven geotextile is subjected to soil pressures.
- Non-woven geotextiles where the fibres are bonded together by various processes such as needle punching or thermal or melt bonding. Non-woven geotextiles can be made using continuous fibres or short (staple) fibres and generally have lower tensile strength, lower modulus, and higher elongation than woven. Non-woven geotextiles generally have a higher permeability, ranging from about 0.04 to 0.01 m/s, but a lower EOS, generally in the range 20 to 500 μm, than woven.
- Composite geotextiles are a combination of woven and non-woven geotextiles, generally have high strength and modulus and low EOS
- Knitted geotextiles are available only in the form of 'socks'. Although relatively strong, knitted geotextiles exhibit high elongation. They have a relatively large but uniform EOS.

Geocells are three-dimensional cellular confinement systems typically made from high-density polyethylene (HDPE) or other durable polymers. They are used to confine and stabilise soil, aggregate, or other infill materials in a grid-like network.

Geomembranes are impermeable sheets made from materials such as polyethylene, polypropylene, or PVC. They are utilised in containment systems to prevent the seepage of moisture/water.

Different types of geosynthetics materials are used for subgrade stabilisation in various forms such as geogrid, geomembrane, and geocell to perform the function of strengthening, separation, filtration, lateral drainage and reinforcement.

In the construction of temporary road works such as haul roads:

1. Geotextiles shall be considered for use as reinforcement in weak subgrades of California bearing ratio (CBR) <3 % (i.e., subgrade class S1) to maximise their benefit in terms of pavement thickness reduction.
2. It is recommended that a minimum of 150 mm of granular material be placed over a geosynthetic, and where design shows that without a geosynthetic, 200 mm of granular material is required on strength grounds only, it is unlikely to be economic to include a geosynthetic.
3. A geogrid shall be placed on top of the geotextile where there is a need to distribute wheel loads over a greater area of the subgrade in order to achieve strength improvements.

For road construction of permanent works over subgrade class S1 and where rut depth is limited to less than 20 mm:

1. Geotextiles shall serve the purpose of separators and shall be placed at the interface of subgrade and sub-base layer to prevent the loss of aggregates into the soft subgrade. The geotextile should have high permeability to allow rapid dissipation of excess porewater pressures that have been built up during transient wheel load passes.
2. Geotextiles shall conform to the requirements of the product standard BS EN 13251 and BS EN 13249.

The following geotextile characteristics (Table 7.8) shall be determined for their suitability in road works for reinforcement, R; Filtration, F; and Separation, S. The obtained test result shall be greater than the material specification provided in the design.

Table 7.8 Relevant Geotextile Characteristics and Test Methods

Characteristic	Test method	Function
Elongation at maximum load	EN ISO 10319	R
Dynamic perforation resistance (cone drop tests)	EN 918	F, R
Damage during installation	ENV ISO 10722 – 1	F, R, S
Characteristic opening size	EN ISO 12956	F
Water permeability normal to the plane	EN ISO 11058	F
Resistance to weathering	EN 12224	F, R, S

Geocells shall be used for subgrade improvement in accordance with the manufacturer's specifications and where a cost saving arising from their use as evidenced outweighs the risk of failure.

Note: *Polyester geosynthetics shall not be used where the sub-base is lime-treated due to the strength reduction of the material arising from coming into contact with calcium hydroxide.*

8 Pavement Materials

8.1 General

Pavement materials can be obtained from within the road reserve, from material borrow sites, and from stationery and moving production plants. The materials include:

1. Natural granular materials (sands and gravels).
2. Crushed stone materials and screened rock.
3. Hydraulically and lime improved materials.
4. Hydraulically bound materials.
5. Bitumen stabilized materials (BSMs).
6. Primes, tack coats and precoating fluids.
7. Bitumen bound materials.
8. Bituminous seals and micro-surfacing.
9. Concrete.
10. Concrete paving blocks.

The choice of the pavement materials and, hence, of the pavement structure, will largely depend on the types and the respective costs of the natural materials locally available.

In light of climate change events, it is important that the designer should identify sections of the road that are vulnerable and select materials appropriately. The measures could include:

i. Provision of a bituminous surface dressing on asphalt pavements to mitigate increased oxidation rates.
ii. Use of polymer modified asphalts.
iii. Use of hydraulically modified granular materials that are less susceptible to moisture.
iv. Use of low plasticity materials to minimise the loss in strength when wet.
v. Use of geo-grids to enhance strength of granular materials, and geo-membranes to cut off capillary rise.

8.2 Sub-base Materials

8.2.1 Natural Gravels

Many different types of natural gravel occur in Kenya, namely lateritic gravels, quartzitic gravels, calcareous gravels, some forms of weathered rock, soft stone, coral rag, volcanic gravels (scoria) and conglomerate.

Natural gravels for sub-base shall have a CBR of at least 30 %, at 95 % MDD (Modified AASHTO) and after 4 days' soak.

In addition, the gravel shall comply with the requirements shown in Chart GM8 (GM7 for LVSR).

The following points should be noted:

I. The maximum Plasticity Index allowed depends on the climate and on the type of base to be placed. Plasticity Indices of up to 20 for G25 and 12 for G30, may be tolerated in dry areas i.e. mean annual rainfall less than 500 mm. On the other hand, the Plasticity Index should not exceed 15 for G25 and 12 for G30 in wet areas under open-textured (permeable) bases, because of the risks of soaking and pumping of plastic fines up into the base.

II. Care is needed in using gravels derived from weathered igneous rock, as these materials are extremely heterogeneous and coarse fragments that appear to be sound may contain minerals that are already decomposed. In particular weathered basic rocks like basalt, phonolite and dolerite are of very poor quality and their use should be as limited as possible. Weathered micaceous rocks, such as micaschists and some types of gneiss, are likely to give rise to similar difficulties.

8.2.2 Clayey and Silty Sands

Various types of clayey and/or silty sands are to be found in Kenya. These are suitable as sub-base materials if their CBR, at 95 % MDD (Modified Compaction) and after 4 days' soak, is at least equal to 30 % and if they comply with the other requirements given on Chart GM7 and GM8. Attention is drawn particularly to the plasticity limitations.

8.2.3 Hydraulically Improved Granular Materials

Natural gravels, sands, and clayey sands, which do not meet the sub-base requirements given on Chart GM7 and GM8, may be improved by treatment with cement or lime.

Non-plastic and low plasticity materials can readily be treated with cement, but do not lend themselves to lime treatment whilst plastic materials can be successfully treated with lime. Treatment with lime appears to be advantageous when the percentage of fines (material passing 0.425 mm size) and the Plasticity Index exceed 15 % and 10 % respectively.

The materials suitable for treatment, the cement, lime, or other hydraulic binder, and the treated material shall comply with the requirements given in Charts HM1 to HM4.

Curing shall be carried out by covering the surface with either approved plastic sheeting, moist soil, straw and/or keeping the surface damp by frequent applications of a light spray of water.

8.2.4 Graded Crushed Stone

Stone is abundant in many parts of the country, and graded crushed stone may be used as sub-base material, particularly where no suitable natural gravel can be economically found.

The material requirements, traffic limitations and construction procedures are summarised in Chart GM11.

The following points should be noted:

I. Crusher-run should be used as much as possible, for obvious economic reasons. The grading envelopes given cover the crusher-runs usually obtained. For some softer stones a grading at the design stage which is coarser than the envelope may be acceptable.

II. Fairly coarse crusher-run (0/60 mm) which is economical may be used for medium and light traffic.

III. The grading to consider is that after compaction and rolling may cause further crushing and produce additional fines in the case of soft stone (LAA > 40).

IV. Six classes of GCS in accordance with Chart GM11, stone classes A, B, C, D, E and F have been defined based on hardness and crushing ratio. Classes A, B, and C stone must be crushed stone, whereas rounded aggregate (alluvial deposits) may be accepted for Class D and lower. Class A stone is required for design traffic class TC30 and TC50, Class B for TC17, Class C for TC3 and TC1, and Class D for TC3 and TC1.

V. The general requirement is that the fines shall be non-plastic for Class A, B, and C, but allowed for classes D, E and F. However, under light traffic and in dry areas, limited plasticity may be tolerated (Plasticity Index not more than 6-8), provided that the stability and the permeability of the material remain adequate. Plasticity requirements are as follows:

- **a.** Class A: Non plastic.
- **b.** Class B: Non plastic.
- **c.** Class C: Non Plastic.
- **d.** Class D: PI ≤ 10.
- **e.** Class E: PI ≤ 10.
- **f.** Class F: PI ≤ 12.

Stone class D, E and F shall be used as gravel i.e., G80 and G30 materials.

VI. To avoid segregation: GCS classes A, B and C should be mixed and moistened in stationary plant and laid by paver. Classes D, E and F should always be kept moist during handling, transportation and laying and should not be stockpiled in heaps higher than 5 m.

VII. All of the following compaction requirements shall be complied with:

- **a.** Reference to the Maximum Dry Density (Vibrating Hammer method):
  - Average dry density: Min. 98 % MDD (V.H.).
  - No result below 96 % MDD (V.H.).
- **b.** Reference to the Specific Gravity of the stone (Oven dry value):
  - Average dry density: Min. 82 % S.G.
  - No result below 80 % S.G.
- **c.** No visible movement under a steel wheeled roller applying at least 5000 Kg/metre width of roll.

VIII. Special care must be taken to ensure that the layer edges are properly compacted.

8.2.5 Dry-bound and Wet-bound Macadam

Macadam is comparable in performance with a graded crushed stone and should be used as substitutes for GCS in standard pavement structures that provide that option. For sub-base it is suitable for traffic application from TC3 to TC150. They are especially useful where good internal drainage is required within the pavement. The materials are also not susceptible to moisture ingress and are therefore good in mitigating capillary rise and for enhancing climate resilience. Aggregate hardness, durability, particle shape and in-situ density requirements are similar to those given for graded crushed stone and are contained in materials Chart GM13 together with particle size distribution requirements.

The material is also suitable where labour-intensive construction is an economic option. They consist of nominal single-sized crushed stone and non-plastic fine aggregate (passing through the 5.0 mm sieve). The fine material should preferably be well graded and consist of crushed rock fines or natural, angular pit sand.

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The dry-bound Macadam process involves laying single-size crushed stone of 75- 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 should be 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 stone should be toward the coarse end of any recommended range. Economy in the production process can be achieved if layers consisting of 50 mm nominal size stone and layers of 37.5 mm nominal size stone are both used. This allows the required total thickness to be obtained more precisely and better overall use made of the output from the crushing plant.

Water-bound Macadam is similar to dry-bound Macadam. It 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 (grouting sand) that passes through 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 needed 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.

8.2.6 Soft Stone

Where soft stone is to be used for the sub-base layer, its contribution to the pavement strength will depend on its quality. A poor-quality soft stone is to be assessed and used in accordance with the requirements for a natural gravel. A good quality soft stone meeting the requirements given below may, however be improved by the use of cement or lime and used for sub-base layers:

I. L.A.A. less than 70 %.

II. P.I. on fines from L.A.A. test: Non-plastic.

III. P.I. of material passing 0.425 mm sieve from the 'as dug' material: Max 15 % and plasticity modulus less than 250.

IV. C.B.R. at 95 % MDD (Modified AASHTO) after 4 days’ soak: Min. 60 %.

8.2.7 Hand Packed Stone

Hand-Packed Stone (HPS) paving consists of a layer of large broken stone pieces (typically 150 to 200 mm thick) tightly packed together and wedged in place with smaller stone chips, rammed by hand into the joints using hammers and steel rods. The remaining voids are filled with sand or gravel.

The structures also require a capping layer when the subgrade is weak and a sub-base of G30 material or stronger is used. Prior to laying of the HPS, edge restraint of either a concrete or stone kerb may be constructed to improve stability.

Stones may be extracted from trachyte, basalt, granite, or hard sandstone rock. The characteristics of the parent rock material for the various applications are provided in the materials chart GM12.

The stones shall be laid by hand and closely packed using stones of maximum size of either 150 mm or 200 mm. The larger stones are packed with the longer dimensions placed vertically with the base at the bottom and the voids in between filled with the smaller stone driven with the apex facing down. The layer shall be proof rolled with a steel wheeled roller with a minimum axle load of 8 tonnes in the presence of the Engineer, who shall approve of its stability before compaction. The materials specifications are provided in Chart GM13.

Compaction shall be by a steel-wheeled roller of at least five tonnes per metre width of roll. It shall consist of four static runs or until there is no movement under the roller. There shall follow vibratory compaction until an average dry density of 85 % minimum of specific gravity of the stone has been achieved. No result shall be below 82 % of specific gravity.

After compaction a filler of crushed rock fines or sand shall be spread over the surface and brushed into the joints. The surface shall be vibrated using a vibratory plate compactor to ensure complete filling of spaces within the hand packed stone surface matrix by the fine aggregates. Where necessary, further sand or fines shall be added and the surface re-vibrated.

The filler shall be free from foreign matter and fines passing 0.425 mm sieve shall be NON-PLASTIC.

8.3 Base Materials

8.3.1 Natural Gravels

Natural gravels for base shall have a CBR of at least 80, at 95 % MDD (Modified AASHTO) and after 4 days' soak. The other material requirements are detailed in Chart GM10. Notably, the plasticity index should not exceed 10. Crushed stone or crusher run not complying with requirements of graded crushed stone, may be used as 'natural gravel', provided it complies with the requirements in Chart GM10. In fact, where available, it is preferred to natural gravel due to its low moisture susceptibility and high shear strength.

Natural gravels meeting these requirements are very scarce in Kenya. Most of the lateritic gravels are not suitable for base, due to their poor nodule hardness (incomplete laterisation) and high plasticity. Weathered rocks are even poorer. Only a few quartz gravels and coral gravels have been found to be adequate.

It may therefore be advantageous to mechanically stabilise unsuitable natural gravels, by mixing in sand to reduce the plasticity or stone (crushed or not) to provide hard coarse particles. An addition of 30 % of sand or stone is regarded, practically and economically, as a maximum.

In other cases, it may be advantageous to obtain relative compaction higher than the usual 95 % MDD (AASHTO T180). However, the bearing strength will be significantly increased by higher compaction degrees only if the coarse particles are hard enough to resist heavy compaction without being crushed or pulverised. In practice, relative compactions up to 97% MDD (AASHTO T180) may be considered.

Natural gravels are not suitable for design traffic classes TC30 and higher, even if they are mechanically stabilised or well compacted, for they are prone to attrition and their characteristics are too variable. However, they may be used for sub-base for TC17 with GCS or BSM base when blended or mechanically stabilised with up to 40% crushed stone aggregates or milled asphalt.

8.3.2 Hydraulically Improved Granular Materials

It is common practice to treat with cement or lime natural materials which have been found to be unsuitable in their natural state. Cement and lime treated materials are thoroughly dealt with in Materials Branch Report No. 343.

It is essential to distinguish between improved materials and stabilised materials.

Improvement consists of treating materials with lime or with a comparatively small amount of cement, so that the engineering characteristics are improved (higher bearing strength, lower plasticity), but the treated material remains flexible.

Stabilisation consists of treating materials with a sufficient amount of cement, so that their cohesion is considerably increased, and significant rigidity is obtained.

Charts HM1 to HM4 covers cement and lime improved materials for base.

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The materials will normally be mixed-in-place, as stationary plant mixing is extremely costly. It has been assumed that the scatter of the stabiliser content may reach ± 30 % of the nominal amount.

As improved materials are fairly flexible, the CBR criterion has been retained.

Cement, and lime improved materials are not considered suitable for heavy traffic because of their generally poor resistance to attrition. Use of this as base layer should be limited to maximum design traffic class TC10 and for sub-base up to TC50. Various types of cement available in Kenya are discussed in section 8.3.2.

It is strongly advised that a mechanical spreader (e.g., spreader box) be used for spreading the cement or the lime, in order to obtain a fairly uniform distribution.

Attention is drawn to the following time limitations:

I. Where cement is used, it is essential that compaction and finishing are completed not later than 2 hours after mixing and that the treated layer is protected against evaporation not later than 4 hours after compaction.

II. When lime is used, the times allowed are 4 hours after mixing and 8 hours after compaction respectively.

In some cases, a prime coat may be too thin to prevent desiccation, in which case protection and curing shall normally be achieved by the application of a bituminous seal coat or prime coat, or by covering the surface with either approved plastic sheeting, moist soil, straw and/or keeping the surface damp by frequent applications of a light spray of water.

No vehicle shall be allowed on a cement treated layer for the first 7 days after compaction.

8.3.3 Graded Crushed Stone

The material requirements, traffic limitations and construction procedures are summarised in Chart GM11.

The following points are stressed:

I. Three classes of stone have been defined, on the basis of their hardness and crushing ratio. Class A, B and C stone must be entirely crushed, whereas Class D, E and F stone may be semi-crushed.

II. Graded crushed stone is not considered suitable for Traffic Class TC30 or higher, or for roads carrying overloaded axles because attrition is probable. Class A stone is required for TC17, Class B for TC10 and Class C for TC3 and TC1.

III. The grading generally required is 0/40 mm. However, for Traffic Class TC17, it is necessary to use 0/30 mm graded crushed stone, in order to minimize segregation and provide sufficient stability.

IV. A minimum amount of fines of 4 % is considered necessary to ensure the stability of the base.

V. No plasticity is allowed in GCS classes A, B, and C.

VI. To avoid segregation, graded crushed stone classes A, B and C should be moistened in a stationary plant and laid by paver. Graded crushed stone classes D, E and F may be laid using a grader. If laid by paver, it should always be kept wet during handling, transporting, and laying and should not be stockpiled in heaps higher than 5 m.

VII. Both of the following compaction requirements for graded crushed stone classes A, B and C shall be complied with:

a. Reference to the Maximum Dry Density (Vibrating Hammer):
   - Average dry density: Min. 98 % MDD (V.H.).
   - No result below 96 % MDD (V.H.).

b. Reference to the Specific Gravity of the stone (oven-dry value)
   - Average dry density: Min. 85 % S.G.
   - No result below 82 % S.G.

VIII. Special care must be taken to ensure that the layer edges are always properly compacted by providing an extra width or specific lateral abutment.

IX. Compaction requirements for graded crushed stone classes D, E, and F shall be comply with compaction to minimum 98% MDD (AASHTO T180).

8.3.4 Dry-bound and Wet-bound Macadam

See section 8.2.5 and materials Chart GM13. It is suitable for use as a base layer for design traffic up to TC30.

8.3.5 Sand Bitumen Mixes

In areas where neither natural gravels nor stone can be found, the treatment of sandy soils may be the only practical solution for base construction. In arid areas, cement treatment has several disadvantages and bitumen stabilisation may then be the most appropriate technique.

Two types of sand bitumen mixes have been identified:

I. Bitumen stabilised silty and clayey sands

The material requirements, traffic limitations and construction procedures are summarised in Chart BB3.

Stable anionic emulsion A3 or fluid medium curing cutbacks (MC 250 - MC 800) may be used. The materials will be mixed-in-place.

Bitumen stabilised silty and clayey sands are suitable for medium and light traffic (TC1 to TC10) but NOT for heavy traffic (classes TC17 or higher).

II. Sand mixes

Chart BB4 gives the material requirements, traffic limitations and construction procedures for sand mixes.

Hard bitumen 40/50 or 60/70 may be suitable. Hot mixing and hot laying are then necessary.

Anionic emulsion A2 and A3 may also be used with the advantage of cold mixing and cold laying.

Single sized sands, such as dune sands, present a very difficult problem because of their low internal friction angle and their high voids percentage. A possible solution is to stabilise them with cement together with bitumen emulsion. The addition of mineral filler or of angular crusher sand to the natural sand will generally be necessary to fill up the voids and to increase the internal friction angle, so that the specified stability is obtained.

Sand mixes are suitable for medium and light traffic. The behaviour of this material under Traffic Class TC30 is still uncertain.

8.3.6 Dense Bitumen Macadam

Dense bitumen macadam is a hot-laid, plant mixture of well-graded aggregate, filler and straight-run bitumen, used for base construction. The resulting mix must conform to stability and flow criteria. Chart BB1 gives the material requirements and construction procedures for dense bitumen macadam.

The following points should be noted:

I. As long as DBM is covered by another layer, harder bitumens (e.g., 30/50 penetration grade) can be used in its manufacture.

II. Dense bitumen macadam is adequate for all traffic but is economically justified only for heavy traffic (TC17 and higher).

8.3.7 Dense Emulsion Macadam

Dense emulsion macadam is a cold mixed, cold laid plant mixture of well graded aggregate and bitumen emulsion and its principal use will be as a pavement overlay material.

The requirements for aggregate and filler are basically the same as for dense bitumen macadam (see Chart BB1). The emulsion can be either slow setting anionic A3 or slow acting cationic K3. The amount of residual bitumen should normally be between 3 and 5 % by weight of the dry aggregate.

Dense emulsion macadam may be a cheap alternative to dense bitumen macadam. Although its strength is only slightly less than that of dense bitumen macadam, no heating or drying of aggregate and no heating of the binder are required. Laying of DBM must only be undertaken by an approved paver. It is pointed out that dense emulsion macadam should not be laid in layers of compacted thickness exceeding 15 mm to permit the evaporation of water.

***Note:** Dense emulsion macadams require heavy compaction to be continued at intervals until all movement ceases; this may not be achieved until a period of some days after laying.*

8.3.8 Bitumen Stabilised Materials (BSM)

Bitumen Stabilised Materials (BSMs) are pavement materials that are treated with either bitumen emulsion or foamed bitumen. The materials treated are normally granular materials, previously cement treated materials or reclaimed asphalt layers.

The use of bitumen stabilised materials (BSM) is preferred for pavement rehabilitation interventions as it allows utilisation of existing pavement materials and immediate passage of traffic. BSM layers are also less susceptible to weaknesses in the underlying layers compared to cement or bitumen bound layers.

The quantities of residual bitumen emulsion or foamed bitumen added do not typically exceed 3 % by mass of dry aggregate. In many situations, active filler in the form of cement or hydrated lime is also added to the mix. The cement content should not exceed 1 % and should also not exceed the percentage of the bitumen stabiliser, (i.e., the ratio of bitumen percentage to cement percentage should always be greater than 1). If this ratio is less than one, then the material should be considered as a cement improved material.

The types of filler used are cement (various types, but not rapid hardening cements), lime, rock flour, fly ash and slag. For the purpose of this Manual, the term active filler is used to define fillers that chemically alter the mix properties. This includes fillers such as lime, cement and fly ash but excludes natural fillers such as rock flour. In this Manual, lime always refers to hydrated lime.

The purpose of incorporating active filler in BSM is to:

1. Improve adhesion of the bitumen to the aggregate.
2. Improve dispersion of the bitumen in the mix.
3. Modify the plasticity of the natural materials (reduce PI).
4. Increase the stiffness of the mix and rate of strength gain.
5. Accelerate curing of the compacted mix.

The addition of bitumen emulsion or foamed bitumen to produce a BSM results in an increase in material strength and a reduction in moisture susceptibility due to bitumen coating of some of the fine aggregates within the materials matrix. Such 'non-continuous' binding of the individual aggregate particles makes BSMs different from all other pavement materials. The dispersed bitumen changes the shear properties of the material by significantly increasing the cohesion value, whilst effecting little change to the internal angle of friction. A compacted layer of BSM will have a void content like that of a granular layer, and not to an asphaltic material. BSMs are therefore granular in nature and are treated as such during construction.

The primary benefits of using BSMs are:

1. The increase in strength associated with bitumen treatment allows a BSM to replace alternative high-quality materials in the upper pavement. For example, a G80 quality material treated with either bitumen emulsion or foamed bitumen can be used in place of an asphalt base, provided it meets the layer requirements, thereby offering significant cost savings.
2. Improved durability and moisture insensitivity due to the finer particles being encapsulated in bitumen and thereby immobilised.
3. Lower quality aggregates can often be successfully used.
4. The typical failure mode of a BSM (permanent deformation) implies that the pavement will require far less effort to rehabilitate when the terminal condition is reached, compared to a material that fails due to full-depth cracking.
5. BSMs are not temperature sensitive, unlike hot mix asphalt. This is because the bitumen is not continuous throughout the mix.
6. Unlike a hot mixed asphalt, BSMs are not overly sensitive materials. Small variations in both the amount of bitumen added and untreated material properties will not significantly change the strength achieved through treatment. This allows the inevitable variability in the recycled material to be tolerated.
7. Traffic disruption and time delays are minimised by working in half widths and opening to traffic soon after completion. The construction and maintenance of diversions is therefore avoided.
8. Pavements showing a wide range of distress types can be effectively rehabilitated, thus eliminating most of the heavy construction traffic that damages newly constructed layers and adjacent access and service roads.

The BSMs are classified into following three classes:

1. **BSM175**: Bitumen emulsion/foamed bitumen stabilised graded crushed stone or reclaimed asphalt – see materials Chart BM3.
2. **BSM100:** Bitumen emulsion/foamed bitumen stabilised graded natural gravel or reclaimed asphalt – see materials Chart BM2.
3. **BSM50:** Bitumen emulsion/foamed bitumen stabilised gravels or clayey/silty sands, stabilised with higher bitumen contents – see materials Chart BM1.

8.3.9 EME (High Modulus Asphalt)

Enrobé à Module Élevé (EME) is hot-mix asphalt consisting of hard, unmodified bitumen blended at high concentrations (up to 6.5 % m/m) with good quality, fully crushed aggregate to produce a mix with low air voids content. EME is designed to combine good mechanical performance with impermeability and durability. Its key performance characteristics are high elastic stiffness, high resistance to permanent (plastic) deformation and fatigue failure, while also offering good moisture resistance and good workability.

Suitable binders for use in the production of EME include 10/20, and 20/30 penetration grade. The material requirements and construction procedures are summarized in Chart BB2.

Pavements comprising EME as the principle structural layer can be employed for design traffic well greater than 50 million ESALs - with due consideration and selection of supporting pavement materials and structure.

The potential for reduced layer thicknesses makes EME also ideally suited for application in urban areas where disruption to subsurface services can be significantly reduced.

EME is suitable for use in the following circumstances:

1. On heavily trafficked routes, particularly where traffic is slow and channelised, such as on major bus routes.
2. In specific pavements subjected to heavy loads such as dedicated truck routes, loading bays and container terminals.
3. In constrained (boxed-in) pavements such as those found in urban and peri-urban areas.

EME should not be used as surface or binder layer since:

- It may be prone to thermal cracking.
- It may not provide a surface texture with sufficient skid resistance.

Where multiple layers of EME are constructed, it is crucially important that good bonding between the two courses is achieved.

The underlying principle of designing EME base layers is to provide an extremely stiff asphalt layer derived mainly from the properties of a hard binder. While the use of hard grade bitumen, in conjunction with suitable aggregate gradings, will inherently be resistant to permanent deformation, adequate resistance to fatigue failure is ensured by a relatively high binder content.

8.3.10 Hydraulically Modified Stone

This consists of GCS that has been improved with cement to enhance stone-to-stone contact and limit segregation. The material requirements and construction procedures are summarised in Chart HM5 and HB1. The common GCS+2 % are as per Chart HM5.

The following points should be noted:

I. The amount of cement normally required is between 1 % and 4 %. The true quantities to meet the strength requirements should be determined through laboratory testing.
II. The traffic limitations of hydraulically modified stone are presented in Chart HM5 and HB1.
III. Mixing is to be carried out in a stationary plant and hydraulically bound stone is to be laid by mechanical paver.
IV. Compaction and finishing must be completed not later than 2 hours after mixing, and the finished layer must be protected against evaporation not later than 4 hours after compaction. Protection and curing shall be achieved by the application of a bituminous seal coat, preferably bitumen emulsion.

8.3.11 Hydraulically Bound Stone (HBS) for HBS6 and HBS9

As hydraulically bound materials are rigid or semi-rigid, the CBR is meaningless. The most convenient strength criterion for such materials is the Unconfined Compressive Strength (U.C.S.).

Hydraulically bound materials are technically suitable for all traffic, but their use is normally not economically justified for light traffic (Classes TC3 and TC1).

It is recommended to use, as much as possible, the stationary plant mixing method, but it should be borne in mind that only low plasticity materials (Plasticity Modulus not exceeding 700) can be properly mixed in a stationery plant.

Mix-in-place can be used alternatively (and has to be used for materials with a Plasticity Modulus exceeding 700). Powerful mixing equipment is then required, the most usual being heavy duty pulvimixers (more than 100 h.p.).

The scatter of the cement content should not exceed + 10 % of the nominal amount when the materials are mixed in stationary plant and + 20 % when the mix-in-place method is employed.

Compaction must be completed not later than 2 hours after mixing and protection against evaporation must be placed not later than 4 hours after compaction.

Protection and curing shall be carried out as indicated in sections 8.2.3 and 8.3.2.

Vehicles shall not be allowed on bound material for the first 7 days after compaction.

The material requirements and construction procedures are summarised in Chart HB2, and HB3.

The following points should be noted:

I. Ordinary Portland cement is generally used. However, because of the shrinkage problems and time limitations, it may be worthwhile developing the use of special slow-setting cements or the use of retardants (coarsely-ground cements, or mixtures of cement and inert filler, etc.).

II. The amount of cement normally required is between 3 % and 5 %. The true quantities to meet the strength requirements should be determined through laboratory testing.

III. Due to its rigidity, hydraulically bound stone must be placed in thick layers (minimum 150 mm, generally 200 or 250 mm).

IV. Hydraulically bound stone is adequate for all traffic but is economically justified only for heavy traffic (TC50 and higher).

V. Mixing is to be carried out in a stationary plant and hydraulically bound stone is to be laid by mechanical paver.

VI. Compaction and finishing must be completed not later than 2 hours after mixing, and the finished layer must be protected against evaporation not later than 4 hours after compaction. Protection and curing shall be achieved by the application of a bituminous seal coat, preferably bitumen emulsion. No vehicles shall be permitted on hydraulically bound stone for the first 7 days.

8.3.12 Hand Packed Stone

See section 8.2.7. However, if used for base, the surface regularity must be strictly controlled. The materials specifications are provided in Chart GM12.

8.4 Surfacings

8.4.1 Bituminous Binders

8.4.1.1 Penetration Grade Bitumen (Straight-Run bitumen)

The common grades used in Kenya are 40/50, 60/70, 80/100, and 100/250 penetration grades. These shall conform to the requirements contained in ASTM D 946. Their specific use is contained within the materials specification charts in section 8.9.6. Other grades may be used where technically justified.

8.4.1.2 Cut-back Bitumen

The common grades used in Kenya are medium-curing cut-backs MC30, MC70, MC250, MC800, and MC3000. These shall conform to the requirements contained in ASTM D2027. Slow-curing (SC) options are available for use where there is a need to extend the curing period, such as when working with materials contain a large quantity of dust. These shall conform to the requirements contained in ASTM D2026. Their specific use is contained within the materials specification charts in section 8.9.6. Other grades may be used where technically justified.

8.4.1.3 Emulsified Bitumen

The common grades used in Kenya include cationic emulsion grades K1, K2 and K3, and anionic emulsions A1, A2, A3, and A4. Cationic emulsions shall meet the requirements outlined in ASTM D3910 whereas anionic emulsions shall comply with the requirements outlined in BS 434-1. The designer is advised to contact relevant emulsion manufacturers since the grades and developments are always evolving. Their specific use is contained within the materials specification charts in section 8.9.5 and 8.9.6.

8.4.1.4 Performance-Grade Bitumen

Pending classification of suitable performance grades for the climatic regions in Kenya, these can be used on projects in Kenya by the approval of the Chief Engineer (Materials). The designer shall be required to submit the selection criteria used in selecting the bitumen for the specific project. Performance-grade specifications shall conform to the requirements contained in AASHTO M 320.

8.4.2 Modified Bitumen

8.4.2.1 General

Modified bitumen are bitumen whose properties have been enhanced by the addition of polymers, rubber, or other chemical additives. Depending on the modifier, they mitigate against early cracking or rutting, withstand heavy traffic application for longer periods, enhance adhesion, and provide better compaction.

Unmodified bitumen has weaknesses: it becomes rapidly more ‘fluid’ at high temperatures, facilitating rutting in asphalt and embedment in surface dressing: and in the intense light of the tropics, it rapidly ages and hardens, accelerating cracking.

Numerous proprietary products are commercially available to ameliorate these characteristics. The products can be categorised as follows:

8.4.2.2 Polymer-modified Bitumen

It is possible to create additives which enhance the good properties of bitumen whilst suppressing the bad properties. Ethyl Vinyl Acetate (EVA), Styrene Butadiene Styrene (SBS) and Styrene Butadiene Rubber (SBR) are three such examples. Polymer modified bitumen shall conform to the requirements contained in BS EN 14023. They all increase the viscosity, and visco-elastic stiffness, of bitumen at high temperatures and increase fatigue life. The consequences of using polymers to suppress the development of rutting and premature bitumen aging are obvious. Polymer modified bitumen properties shall comply with the requirements of AASHTO M320 and M332 specifications.

8.4.2.3 Rubber-modified Bitumen

Crumb rubber can be added to bitumen to enhance its resilience. This can be obtained by grinding used vehicle tyres, although suppliers of ready-made crumb rubber exist. At up to 18% m/m of the neat bitumen, crumb rubber has been shown to enhance the resilient modulus of asphalt by up to 50%. Its use in Kenya road projects should be considered as part of performance design. Rubber modified bitumen shall meet the specifications of ASTM D6114.

8.4.2.4 Chemical Additives

Elemental sulphur and manganese have both been used as additives. Sulphur becomes a liquid at a temperature greater than ≈120 °C and is added to improve workability at high temperature. The modified shall still have to comply with the requirements of AASHTO M320 and M332 specifications.

8.4.3 Prime Coat, Tack Coat, and Precoat

8.4.3.1 Prime Coat

A prime coat is an application of low viscosity bituminous binder to an absorbent surface. Its purposes are to waterproof the surface being sprayed and to help bind it to the overlying bituminous course. The use of prime emulsion is preferred. These can be obtained from bitumen manufacturers and suppliers. A3 Anionic emulsion diluted with water can be used.

All non-bituminous base shall be primed.

The most appropriate binders for priming are medium curing fluid cut-backs MC 30 and MC 70.

MC 30 is suitable for practically all types of materials. For open-textured surfaces like graded crushed stone, MC70 is recommended.

The rate of application will depend on the texture and density of the material to be primed. It is usually between 0.8 and 1.2 l/m^2.

It is good practice to dampen the surface to be primed as this facilitates the penetration of the binder.

It may happen that priming a cement treated layer with cut-back causes slight surface disintegration, because of interference with the cement hydration. The effect of cut-backs on cement treated materials should always be assessed on preliminary trial sections. If difficulties arise, priming should be replaced with a bitumen emulsion tack coat.

If the prime coat has to be trafficked before the surfacing is placed, it should be blinded with clean, non-plastic natural sand, crusher dust or fine aggregate. However, this should be done after the prime has achieved 10 mm penetration and water or binder has fully evaporated.

8.4.3.2 Tack Coat

A tack coat is a light application of bituminous binder to a bituminous or concrete surface. Its purpose is to provide a bond between the surface being sprayed and the overlying bituminous course.

The following types of binders may be used:

1. Medium curing cutbacks (MC250, MC800 or MC3000).
2. Quick breaking emulsions (A1 or K1).
3. A3 Anionic emulsion diluted with water.
4. Polymer-modified emulsions.

The rate of spray will depend on the surface texture. It is usually between 0.3 and 0.8 l/m^2 residual bitumen.

For very thin applications, emulsion would normally be preferred, as it can be diluted with water, permitting a thin film of bitumen to be achieved at a convenient rate of spray from the distributor. All tack coats should be applied to a cleaned surface shortly before laying the next bituminous layer but allowing sufficient time for evaporation of cutter or run-off of emulsion water.

8.4.3.3 Precoat

Precoating of aggregates may help to improve the aggregate-bitumen bond. This is especially effective for aggregates that are dusty or that may be used in a dusty environment. Aggregates may be pre-coated using a special formulated emulsion or a mix of 45-60% penetration grade bitumen and 55-40% by volume of Kerosene.

8.4.4 Surface Dressing

8.4.4.1 General

Surface dressing is a surfacing of the highest importance. It can be used as a definitive surfacing under light and medium traffic or as an economical first surfacing under heavy traffic. Further, surface dressing can be applied as a maintenance process; the service life of all forms of bituminous surfacing is extended by periodic resealing.

Chart SU1 gives the material requirements, traffic limitations and construction procedures.

The design method (i.e., choice of chippings size, choice of binder and determination of application rates) is detailed in Charts SU1.

8.4.4.2 Type of Binder

Surface dressing can be used on roads carrying very heavy traffic. However, classical binders like cut-backs and emulsions are suitable only for traffic up to about 6,000 vehicles/day on 2 lanes. For heavier traffic, or high stress areas such as junctions and steep sections, special types of binder, for example those modified by the addition of polyvinyl chloride or rubber, are required.

Three main types of binder can be used for traffic up to 6,000 vehicles/day, these are:

1. Medium curing cut-back 3000 (Site blended from 80/100 penetration bitumen and 5-10 % Kerosene).
2. 80/100 penetration grade bitumen (Neat or site blended with up to 5 % Kerosene for workability depending on road surface temperature).
3. Cationic emulsion (K1-70).
4. Polymer-modified binders.

Section 8.7 discusses the most suitable binder in various climatic areas and for different aggregates. It can be seen that cationic emulsion is preferred in wet areas.

As regards cutbacks, the governing feature is the road surface temperature at time of application. Section 8.7 presents the road temperature/viscosity relationship for several bituminous binders and will help select the most suitable binder.

Medium curing cutbacks of appropriate viscosity may also be obtained by blending straight-run bitumen with diesel or kerosene, should the commercial product not be available.

8.4.4.3 Chippings Class and Size

Four classes of chippings have been defined based on their hardness, shape and their respective conditions of use are as follows:

a. *Class 1* where the total traffic exceeds 6,000 vehicles/ day.

b. *Class 2* where the total traffic is 2,000-6,000 vehicles/day.

c. *Class 3* where the total traffic is 500-2,000 vehicles/day.

d. *Class 4* where the total traffic is less than 500 vehicles/day.

The total traffic referred to above is the average daily traffic during the year of application of the surface dressing and which includes cars and light goods vehicles as even they have a wearing effect on surface dressing.

For double seals, the first layer is usually made up of 14-mm chippings followed by a second layer of 7 mm chippings. The use of 20 mm chippings is generally not justified for double seals (except under very heavy traffic on soft surfaces). However, for triple seals, the 20 mm chippings form the first layer, the second being 14 mm chippings, and the third seal being 7 mm chippings.

The design of surface dressing is discussed in section 8.7.

8.4.4.4 Number of Seals

Double surface dressing is normally recommended for new roads. Triple surface dressing is justified only where complete imperviousness must be guaranteed for a number of years and where there is heavy traffic on steep slopes or tight curves.

Single surface dressing should not be used for new construction, but only for maintenance.

8.4.4.5 Racked-in Surface Dressing

Racked-in surface dressing should be used when greater interlock is required, such as in high traffic acceleration or deceleration, or on tightly knit surfaces. Racked-in surface dressing is designed as a double surface dressing, but the chipping spread rate of the first seal is reduced by 10 %. During construction, all the bitumen for the first and second seal are applied in one pass. Then the chippings for the first seal applied, then the chippings for second seal applied, then rolling is done. It may also be used if assurance of deep seal is required particularly in high rainfall areas and in this case a second seal will be necessary to protect the second stone spread.

8.4.4.6 Inverted Surface Dressing

This is used on fine-textured (tightly knit) bases such as hydraulically stabilised or modified bases, and lateritic bases. It is designed as a double surface dressing, but during construction, the 'second seal' is constructed first, then the 'first seal' is applied on top.

8.4.4.7 Adhesion

Clean dry chippings should be used with possibly an adhesion agent with cut-back binders. The chippings should be clean and damp if a cationic emulsion binder is used. If cutbacks are used with damp chippings an adhesion agent will probably be required.

8.4.5 Other Bituminous Seals and Seal Combinations

8.4.5.1 Slurry seal and Cape Seal

A slurry is a mixture of slow-breaking bitumen emulsion (A4-60 or K3-65), sand, fine crushed stone aggregates, cement, and water in pre-determined proportions. The specification of materials required to produce good-performing slurry are contained in Chart SU2. A slurry seal as a first seal is generally only advisable for low volume roads of class TC0.01, and not higher. Slurry is produced and laid using a special purpose-built machine, however labour-based methods of mixing and laying also exist, although they are generally slow.

A Cape Seal is simply an application of a slurry seal on top of a single surface dressing. It is known to perform very well, even at high volumes of traffic.

8.4.5.2 Cold Mix Asphalt

Cold mix asphalt for surfacing is a cold mixed and cold laid mixture of well-graded aggregates and bitumen emulsion.

The emulsion can either be slow setting anion A3 or slow acting cationic K3 with at least 60% residue binder content. The amount of residue bitumen should normally be between 6.0 to 7.5%.

Voids in the mix shall be between 3 to 8%. If more voids are present based on the mix design, then a fog spray of K1-70 and quarry dust of 0/6 mm shall be provided to seal the surface.

For labour intensive construction methods, the bitumen emulsion and aggregates are usually mixed in pans and placed on the road using labour-based methods. The compaction is then applied by rollers until the surface has attained stability – usually within a few minutes.

Materials specifications, construction procedures and tolerances for surface dressing are presented in chart SU3.

8.4.5.3 Otta Seal

Otta seal is a bituminous seal constructed by the application of a thick application of relatively soft bitumen (MC800, MC3000, 150/200 penetration grade, or aggregate-specific bitumen emulsions) followed by the application of graded granular aggregate.

The cover aggregate shall consist of natural gravel, graded crushed stone or a mixture of these. The surface is then rolled using a pneumatic-tyred roller. After continuous rolling and traffic action, the bitumen works its way up through the aggregate interstices. Over time, a tight impervious surfacing results. Otta Seal has been used hitherto in Kenya since the 1980s, and it has shown good performance.

The major advantages of Otta Seal are that it can be constructed using crushed aggregates or natural screened aggregates; they can be constructed with or without priming the base layer and have long durability.

Otta seals are designed according to the guideline, “A guide to the use of Otta Seals, 1999; NPRA Publication No. 93.”

Materials specifications, construction procedures and tolerances for surface dressing are presented in chart SU4.

8.4.5.4 Sand Seal

A sand seal is an application of a bituminous binder on a primed road base covered with natural sand, fine crushed rock aggregates or a mixture of both. This should not be used on roads other than low-volume roads, except as a maintenance intervention.

The sand or fine aggregate shall be free from organic matter, clayey and other deleterious material.

Sand seal application is usually limited to roads carrying light traffic of less than 100 vehicles per day if it is applied as a first seal, or less than 500 vehicles per day if it is applied as a second seal on Surface Dressing, Otta Seal or Cold Mix Asphalt.

Sand seals should not be used on steep grades greater than 6 %. In addition, the use of single sand seal directly applied on the base layer is strongly discouraged. It should always be constructed as a double seal, unless it is applied as a second seal on another type of surfacing.

Sand seal is not designed in the same sense that a surface dressing can be designed. The design is in fact a recipe design. The sand should be applied at a rate of 6 to 7 x 10^{-3} m^3/m^2. The binder, which may be a cut-back (preferably MC800 in cold areas and MC3000 in hot areas) or an emulsion (K1-60 or K1-70), should be spread at a rate of approximately 1.0 to 1.2 kg/m^2.

Materials specifications, construction procedures and tolerances for surface dressing are presented in chart SU5.

8.4.5.5 Surfacing Combinations

The following surfacing options are suitable for low-volume sealed roads. They should be used in the following recommended layer combinations to prevent premature failures. Experience has also shown that the following combinations work well and are usually cost-effective:

1. Double Surface Dressing.
2. First layer of Single Surface Dressing followed by a second layer of Sand Seal.
3. First layer of Single Surface Dressing followed by a second layer of Slurry Seal.
4. First layer of Single Otta Seal followed by a second layer of Sand Seal.
5. First layer of Cold Mix Asphalt followed by a second layer of Slurry Seal.
6. First layer of Cold Mix Asphalt followed by a second layer of Sand Seal.

In each case, each layer is designed as described under the respective seal. In addition, ample time should be given for th**e first seal to cure before applying the second seal.**

8.4.5.6 Road Oiling

Road oiling is primarily used for dust control. Its main function is to protect the base layer materials from being abraded by traffic. Road oiling should be treated as a temporary measure before applying a more permanent bituminous seal.

Substances used for road oiling include slow-curing cut-backs SC70 or SC250, used motor oil, lignin-sulphates, propriety products and polymers, vegetable oils, molasses, and other substances.

The 'oil' is sprayed on the road surface by a distributor or purpose made sprayer. The spray rate of the substance is often provided by the supplier. For slow-curing cut-backs, the application rate is usually 3 to 4.5 l/m^2. This method works best on porous road bases.

Anionic bitumen emulsion (A4) and medium-curing cut-backs MC250 and MC800 may also be used by mixing in place base materials; that is if the plasticity of the base material is less than 15 % and the percentage of fines passing 75 μm is less than 30%. The treated thickness usually varies between 50 and 75 mm.

It must be noted that some substances used for road oiling in the past are now not environmentally acceptable, as they may be leached or washed into open water bodies by the action of rain. Consultation must be made with the environment office before using any road oil. Moreover, road-oiling must be considered as a temporary measure and must be sealed soon afterwards for economic value to be realised from the process.

8.4.6 Functions and Performance of Bituminous Premix (Asphalt)

8.4.6.1 General

Bituminous premixes are generally referred to as asphalt. This is a combination of bitumen and aggregate proportioned in a manner to provide appropriate performance economically. During production, the constituents are premixed together in a batching plant, transported, and placed as a mat more than 30 mm) using a paver; hence the name 'premix'.

8.4.6.2 Functions of a Premix Surfacing

The road user mainly requires a premix surfacing to:

1. Provide a satisfactory riding quality.
2. Provide a sufficient skid resistance under all weather conditions.

The design engineer requires a premix surfacing to:

1. Protect the underlying pavement layers from ingress of water and the abrasive and disruptive actions of traffic.
2. Have a maximum maintenance - free life (this implies that the surfacing itself is not subjected to excessive strains).
3. Fulfil certain economic conditions.

8.4.6.3 Factors Influencing the Performance of Premix Surfacings

The performance of a premix surfacing depends on:

1. The local climatic conditions.
2. The intensity and distribution of traffic.
3. The properties of the supporting pavement layers, in particular their stress-strain characteristics.
4. The bonds between the layers.
5. The engineering characteristics of the bituminous mixture (stability, durability, fatigue resistance, permeability, etc.).

8.4.7 Asphalt Surfacings

8.4.7.1 General

From a structural point of view, it is absolutely essential to make a clear distinction between rigid and flexible mixes. Rigid (or high stability) mixes are designed to resist rutting and high stresses, whereas flexible mixes are designed to resist comparatively high flexural deformation.

From a practical point of view, it is usual to differentiate between two basic types of mix:

8.4.7.2 Interlocked Aggregate Mixes

These are continuously graded mixes that derive their stability from a good aggregate interlock, obtained by careful adjustment of the mix grading, and from the cohesion provided by the bitumen.

8.4.7.3 Mortar Type Mixes

These mixes derive their stability from the cohesion of a sand-filler-bitumen mortar. They may consist of a mortar only (sand asphalt) or of a mortar plus coarse aggregate that acts as an extender (gap graded asphalt).

In practice, the following types of mix are the most suitable and will cover virtually all the needs of this country:

8.4.7.4 Asphalt Concrete (Continuously Graded Asphalt)

It consists of a well graded mixture of coarse aggregate, fine aggregate, and filler, bound together with straight-run bitumen.

Three types of asphalt concrete have been defined:

i. ***Asphalt Concrete Type I (High Stability)***

   This is a fairly stiff type of mix, designed to resist rutting and high stresses. It can be used as the Wearing Course of a surfacing (Chart SU9) and the Binder Course (Chart 10) of a surfacing.

   It may be placed in a thin layer (50 mm or less) on only rigid or semi-rigid pavements or in a thick layer (minimum 75 mm) on a flexible pavement.

ii. ***Asphalt Concrete Type II (Flexible)***

   This is a flexible type of mix, designed to resist comparatively high flexural deformation.

   It must be placed in a thin layer (maximum 50 mm).

   The material requirements, traffic and use limitations and construction procedures concerning asphalt concrete (Types I and II) are summarised in Chart SU8 and SU9.

   The following points should be noted:

   1. The desired rigidity (or flexibility) will be obtained by the proper combined choice of the following factors:
      a. Grade of bitumen.
      b. Crushing Ratio of Coarse aggregate.
      c. Angularity of sand.
      d. Mix grading.
      e. Amount of filler.
      f. Amount of bitumen.
      g. Filler bitumen ratio.
      h. Voids in total mix.
   2. The production of flexible asphalt concrete Type II (Flexible) will generally require the use of an appreciable proportion of rounded sand and a comparatively high amount of bitumen.
   3. It is important to bear in mind that the mix composition should strike a balance between the requirements of stability and durability. In particular, it is not desirable to achieve Marshall stabilities much higher than the minimum values given in Chart SU8, as this would produce a lean mix prone to rapid hardening of the bitumen and fatigue cracking.
   4. Because of the rapid ageing of bitumen generally observed in Kenya, all asphalt surfacing (especially those made with hard bitumen grades) should be followed by application of surface dressing. Overheating of the aggregate and of the bitumen must be avoided, as it causes oxidisation of the bitumen.

5. Asphalt Concrete Type I (High Stability), because of its low voids content, is very sensitive to mix variations. Furthermore, as the bitumen film is thin, the mix is sometimes difficult to work and compact. This type of mix requires very strict control of production, laying and compaction.
6. Three classes of coarse aggregate have been identified on the basis of hardness and shape. 'Class a' is required for design traffic class TC50 and higher; 'Class b' for TC30, TC17, and TC10; and 'Class c' for TC3 and TC1.
7. Asphalt Concrete Type I is suitable for all traffic, but for traffic classes TC10 and higher, it must be placed on high strength bases (e.g. DBM, HBS9) to minimise premature cracking.
8. Asphalt Concrete Type II is suitable only for medium traffic class TC3 and low volume roads (TC1 and lower). Especially on urban roads, where the traffic flow is composed of mostly light vehicles. If the percentage of load carried by axles loaded to 13 tonnes and higher, expressed as proportion of the total mass of all vehicles surveyed, is equal or greater than 10%, then AC Type I should be used at these design traffic classes.
9. The absolute minimum thickness of asphalt concrete that can practically be laid by the current types of pavers is 25-30 mm, provided the maximum size of aggregate does not exceed 10 mm.

iii. *Superpave Asphalt*

This is a fairly stiff type of mix, designed to resist rutting and high stresses.

It may be placed in a thin layer (50 mm or less) on only rigid or semi-rigid pavements or in a thick layer (minimum 75 mm) on a flexible pavement.

The nominal maximum aggregate sizes (NMAS) are 37.5 mm, 25 mm, 19 mm, 12.5 mm and 9.5 mm. The physical properties of aggregates are as per AC I.

The Superpave definition of NMAS is one sieve size larger than the first sieve to retain more than ten per cent of the aggregate. Gradation classification: the combined aggregate shall be classified as coarse graded when it passes below the control sieve and all other gradations shall be classified as fine graded. All gradations shall be plotted in power chart in accordance with AASHTO M323

Mixes identified for compaction trials shall be manufactured to the laboratory design bitumen content and two other bitumen contents of + 0.5 % and + 1.0 % additional bitumen. Cores will be cut to determine the density of compacted material. The core will then be reheated to 145 ± 5 °C in the appropriate mould and compacted to refusal in the vibrating hammer test. The cores cut from the compaction trial must have a density equivalent to 95 % refusal density.

8.4.7.5 Gap Graded Asphalt

Gap graded asphalt, as the name implies, has a certain range of particle sizes missing from the total aggregate grading. It generally consists of aggregate of a fairly uniform size blended with sand and filler.

As there is very little stone-to-stone contact in the compacted mix, the stability is derived from the cohesion of the sand-filler-bitumen mortar.

The material requirements, traffic and use limitations and construction procedures concerning gap graded asphalt are summarised in Chart SU7.

The following points are to be noted:

1. The gap graded mixes specified in this Manual are designed for thin wearing courses. (25−50 mm).
2. These mixes are impermeable, because of the comparatively high bitumen content and the good distribution of the voids structure.

3. The maximum stone content is limited to 55 % by weight. 'Low stone content' mixes (less than 40 %) are easy to work and compact and are very tolerant to mix variations, whereas 'High stone content' mixes (more than 45 %) become sensitive to changes in bitumen content.
4. The main advantages of this type of mix are its flexibility, its fatigue resistance, and its durability, due to the good distribution of the voids structure and the rounded shape of most of the fine aggregate.
5. 60/70 grade bitumen should normally be used to provide sufficient cohesion. Nevertheless, the use of 80/100 bitumen should not be precluded (bearing in mind the rapid hardening of bitumen generally occurring in this country).
6. The gap graded mixes specified in Chart SU7 are suitable for light to medium traffic (TC10 and lower) and thin wearing courses.
7. Gap graded asphalt may be suitable for heavier traffic or thicker layers, subject to its compliance with more severe specifications, to be derived from further research.
8. As regards thicknesses, the absolute minimum is 25-30 mm. The maximum size of the coarse aggregate and the stone content of the mix shall be adapted to suit the thickness of the layer.

8.4.7.6 Sand Asphalt

Sand asphalt consists of natural sand plus, in some cases, mineral filler and a small proportion of crushed fine aggregate, bound with straight-run bitumen.

The material requirements, traffic and use limitations and construction procedures are summarized in Chart SU6.

The following points should be noted:

1. Sand Asphalt is suitable for wearing course in a thin layer (max. 50 mm).
2. Because of its relative richness in binder and the good distribution of the voids structure, sand asphalt is impermeable, flexible and has a good fatigue resistance.
3. As its resistance to rutting is not very high, sand asphalt is suitable only for light traffic (TC10 and lower).

8.4.7.7 Stone Mastic Asphalt

Stone mastic asphalt (SMA) is a stone-to-stone asphalt structure made up of particles larger than 2 mm, held together by a stiff mastic (bitumen + fine aggregate + filler). This should be used only for traffic levels TC50 or higher. Cellulose fibre must be added as a constituent of SMA with dose rate of 0.3 – 0.5 % by mass of the mixture. The binder for use in SMA should be strictly polymer modified.

SMA possesses a number of technical advantages over conventional asphalt concrete. These include:

1. Enhanced skid resistance.
2. Low permeability (good for climate resilience).
3. Low noise levels.
4. Good rut-resistance if well-designed.
5. High resistance to reflection cracking.

Specifications are given in Chart SU11.

8.4.7.8 Other Types of Mixes

It is thought that the above types of mix cover all the needs of this country.

The use of open-graded asphalt is not recommended as surfacing imperviousness is generally necessary and due to its high air permeability; the mix performance would be adversely affected by the rapid ageing of the binder.

Grouted stone (or penetration macadam) is not favoured either as it requires about 30% more bitumen than the same thickness of plant mix, the construction is complicated, and the final surface is always irregular.

8.4.8 Interlocking Cobblestone Paving

Cobble or dressed stone surfacing consists of a layer of roughly rectangular to cubical dressed stone laid on a bed of sand or fine aggregate, within mortared stone or concrete edge restraints. The thickness of the sand bedding layer shall not be less than 25 mm nor more than 50 mm. The individual stones should have at least one face that is fairly smooth, to be the upper or surface face when placed. Each stone is adjusted with a small (mason's) hammer and then tapped into position to the level of the surrounding stones. Sand or fine aggregate is brushed into the spaces between the stones and the layer then compacted with a non-vibratory roller. Cobble stones are generally 100 mm thick. These options are suited to homogeneous rock types that have inherent orthogonal stress patterns (such as granite), which allow for easy break of the fresh rock into the required shapes by labour-based means.

Cobble stones may be extracted from trachyte, basalt, granite, or hard sandstone rock. Suitable stone must meet minimal standards with regards to water absorption, crushing strength and specific gravity.

Material for the sand bed or the laying course and joint filler should be naturally occurring clean sand or crushed rock fines, free from clay, lumps, or other deleterious material, with a grading curve falling within the envelopes.

Detailed specifications of the cobblestone materials, laying course, and joint filler material are provided in Chart PB1.

For adequate performance of cobblestones, it is essential that they are laid on a suitable base material (as presented in the standard pavement structures in RDM 3.4).

8.4.9 Concrete Block Paving

Block paving is the term applied to flexible surfacing, consisting of precast interlocking concrete paving blocks laid on a laying course. The laying course is the layer of material on which paving blocks are bedded. A firm edge support is required to prevent ravelling at the edge of the surface. Supplementary drainage measures are necessary to prevent saturation of the underlying roadbed through the ingress of water.

Concrete block surfaces are not suitable for sections where the speed of heavy vehicles exceeds 60 km/h. They are most suitable for heavy vehicle parking areas and pedestrian walkways.

Interlocking of the blocks improves strength and durability characteristics to resist the punching loads and horizontal shear loads caused by the manoeuvring of heavy goods vehicles. Interlocking concrete pavement blocks are resistant to oil spillages and may be used in a variety of heavily stressed locations, including intersections, weigh bridges, customs and toll barriers, service station forecourts, container and bus terminals, ports, and bulk cargo handling areas.

In order to ensure durability, interlocking blocks in heavily stressed locations must be of consistently high quality and dimensional accuracy, implying mass production under factory conditions. Precast concrete blocks are manufactured in compliance with KS 827 and are graded in three strength categories: heavy duty, medium duty, and light duty.

Material for the sand bed or the laying course and joint filler should be naturally occurring clean sand or crushed rock fines, free from clay, lumps, or other deleterious material, with a grading curve falling within the envelopes.

Detailed specifications of the paving blocks, laying course and joint filler material are provided in Chart PB2.

8.4.10 Choice of Surfacing

The choice of a type of surfacing will be governed by the structural requirements, road safety requirements, materials availability, and cost considerations.

For heavy and very heavy traffic (Classes TC17-TC150), Asphalt Concrete Type I will normally be required. For medium traffic (TC10), Asphalt Concrete Type I will normally be required. A surface dressing must be applied on top of the AC.

For light and medium traffic (TC3 and lower), surface dressing should normally be chosen, owing to its low cost and good performance (flexibility, imperviousness, and good skid resistance). Asphalt Concrete Type II should also be considered, especially in urban environments.

It is only where no suitable stone for surface dressing is available that the use of Asphalt Concrete Type II, gap graded asphalt or sand asphalt should be considered.

8.5 Recycled Asphalt Pavement (RAP)

8.5.1 Current Practice in Kenya

It is economically viable, and environmentally desirable, to reclaim worn-out asphalt and granular pavements, re-process, and re-use them in the rehabilitated road.

The stockpiling of RAP prior to re-processing is a critical part of the procedure. It is important that the variability of the RAP is well-controlled. Stockpiled RAP tends to agglomerate, and a crust forms, depending on the hardness of the bitumen and the ambient temperature. To offset this tendency, it has been found that the larger the stockpile the better. RAP readily absorbs moisture, and it is best stored under roofing in an open-sided building.

8.5.2 RAP in Improved Subgrade Layers

An improved subgrade would only be used in the reconstruction of a road pavement where (in the unlikely event) the in-situ subgrade CBR is <5 %. The milled asphalt material should meet the grading specification in Table 8.1. The capping layer can consist of 100 % of RAP providing the bitumen content is <10 %. The recycled material can be laid to a maximum thickness of 200 mm providing the required density is obtained.

Table 8.1 Grading for RAP for Use in Improved Subgrades

BS Sieve Size (mm)	% Passing Sieve Size
75	65 - 100
37.5	45 - 100
10	15 - 60
5	10 - 15
0.6	0 - 25
0.063	0 - 12
0.6	0 - 25
0.063	0 - 12

8.5.3 RAP in Improved Subgrade Layers

The quality of the aggregate in the RAP should at least meet the requirements for this layer. Fresh aggregate can be added to modify the grading and the compacted layer of the blended material should be acceptable providing the bitumen is hard enough not to hinder compaction and enable the required moisture content, which is between the optimum moisture content and -2 % of optimum moisture content achieved with the BS Vibrating Hammer test, BS 1377, Part 4, 1990 to be met. This is also subject to a Trafficking Trial, whereby the compacted RAP is laid on a prepared trial area constructed to a specified standard and then trafficked with a loaded truck until 1000 ESA have been applied. The mean deformation in the wheel paths must be <30mm for the material to be acceptable.

8.6 Asphalt Mix Design

8.6.1 Aggregate Blending by Direct Proportioning

Often, multiple stockpiles of aggregate are blended to meet the final specified requirements. A washed sieve analysis must be performed on every aggregate stockpile in order to calculate the final aggregate blend in the mixture to be designed.

Calculating a blended gradation, assuming all aggregate fractions have a similar Bulk Specific Gravity (Gsb):

$$P = (A \times a) + (B \times b) + (C \times c) + \ldots$$

Where,

P = the blended percent passing for a given sieve

$A,B,C,$ = the percent passing a sieve for an individual stockpile

$a,b,c,$ = proportion of stockpile to be added in the blend, where total = 1.00.

The above-mentioned gradation and blending operations result in an aggregate size distribution based on percentage of mass. Volumetric properties such as air voids and VMA are directly impacted by the amount and size of aggregate particles and the resulting packing characteristics in the final mixture. However, when the specific gravities of the individual aggregates differ or vary significantly (by 0.20 or more), the blended gradation, based on the mass of the aggregates, may have different volumetric characteristics when compared to an equivalent gradation of materials having similar specific gravities. For such cases, prior experience with similar aggregates, and a trial-and-error approach should be used.

8.6.2 The Bailey Method of Aggregate Blending

The Bailey Method was developed to prevent rutting while also maintaining the durability of mixtures, and to better understand the mechanics of aggregate packing and its contribution to the compressive strength and stability of asphalt pavements. It is applicable to coarse-graded, fine-graded and SMA mixtures. The method can help explain why some Superpave mixes are difficult to compact. It also provides insight as to why small gradation changes, which often occur during production and are within allowable tolerance can cause significant changes to mixture volumetric properties and/or field compactibility.

The Bailey Method is based on the principle that maximum compressive strength of an asphalt mix is obtained when there is stone-to-stone contact between as many aggregate particles as possible.

The Bailey Method provides a good alternative for the design of rut-resistant mixes capable of passing wheel tracking tests and refusal density. The approach also serves as a useful aid when adjusting at the plant to improve air voids, VMA and the overall workability of the mix.

Further details can be found in Appendix C.

8.6.3 Asphalt Concrete Mix Design Process

Ideally the design of an AC 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.

iv. All asphalt mixes shall be subjected to water sensitivity tests by way of indirect tensile strength tests (ratio of wet to dry ≥ 0.8)

Mix design for asphalt materials using the Marshall Method and the Superpave Mix Design Method shall be based on the recommendations given in the Asphalt Institute Manual Series, MS-2.

The mix design of asphalt shall be undertaken using the following methods (Table 8.2).

Table 8.2 Mix Design Methods for Design Traffic Classes

Design Traffic Class	Mix Design Method	Additional Notes
Up to TC3	Marshall at 75 blows.	Special consideration for severe sites.
TC10 and TC50	Marshall and Vibrating Hammer. The mix design may make use of the Superpave aggregate gradation. The Bailey method (described in the following section) may also be used for blending the aggregates.	Consider the use of polymer-modified binders.
TC80 and TC150+	Performance design is recommended at this level. In lieu of that then: Marshall + Vibrating Hammer+ Gyratory compaction with Wheel Tracking, Roller slab compaction with Wheel Tracking, or Superpave. In addition, flexural beam fatigue tests, and repeat load creep tests may be requested by the design engineer	Consider the use of polymer-modified binders or Rigid Pavements.

Notes: *Please see Appendix B, C, D, and E for further guidance.*

Severe sites

These are sites or sections where any of the combination of the following can occur:

1. Slow-moving commercial vehicles.
2. Steep gradients (both uphill and downhill) of slope greater than 6 %.
3. Where heavy vehicles are stationary e.g., at junctions, police check points, weighbridges, etc.
4. Roundabouts and tight curves.

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 Asphalt Institute MS-2). A Marshall design should be carried out on the selected mix but with no aggregate larger than 25 mm.

If the Marshall requirements are satisfied then coarse aggregate between 25 mm and 37.5 mm or 25 mm and 28 mm, 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.

It is recommended that AC 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 50 mm, for 0-20mm AC or 19mm NMS for Superpave grading, to more than 100 mm.

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 12697-32).

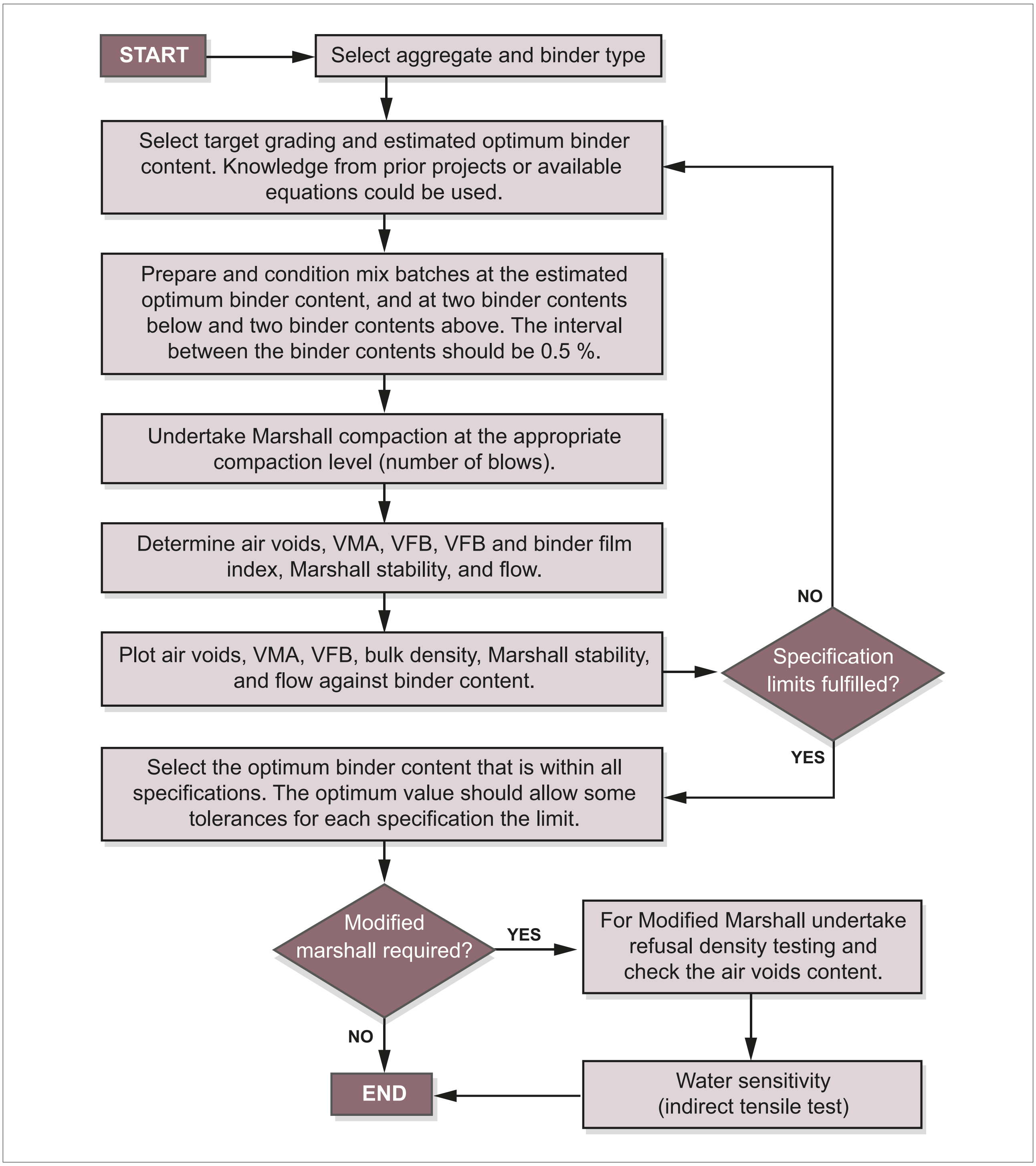
The test moulds for this method are large enough to allow the design of mixes containing aggregate particles larger than 25 mm. The apparatus is easily transportable and can be used to compact hot mix samples anywhere on site provided a suitable power source is available.

The design bitumen content is determined by compacting samples to refusal using the method described in Appendix B. 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 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 B).

A flow chart summarising the mix design procedures are as shown in Figure 8.1 (Marshall), Figure 8.2 (Superpave) and Figure 8.3 (performance testing). This approach is also applicable to the design of fine-grained mixes (e.g., sand asphalt).

Figure 8.1 Mix Design procedure for Dense-graded Mixes (AC, DBM) by Marshall Method



1 2 3 4 5 6 7 8 Pavement Materials

Figure 8.2 Mix Design Procedure for Dense-graded Mixes (AC, DBM) by Superpave Method

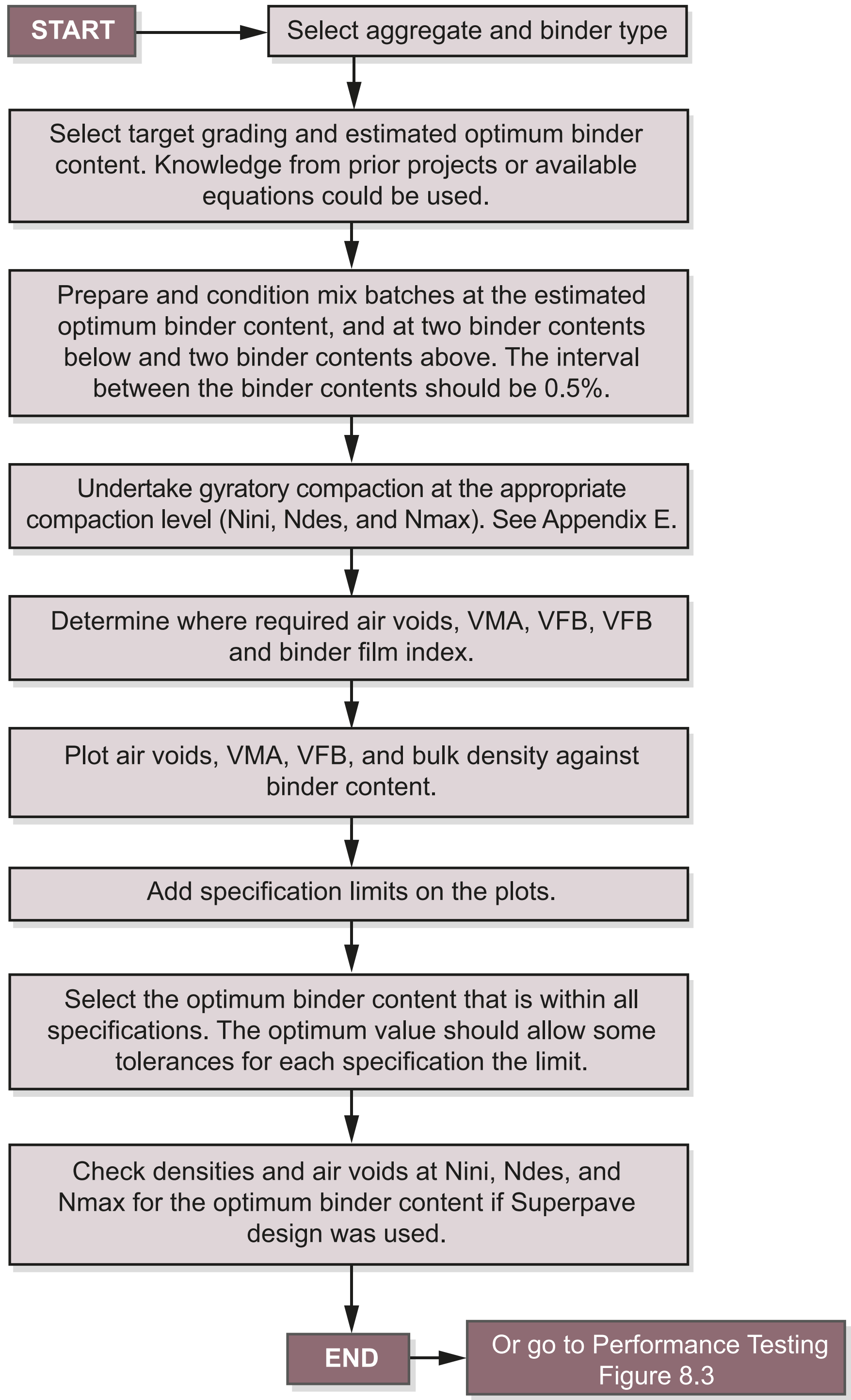
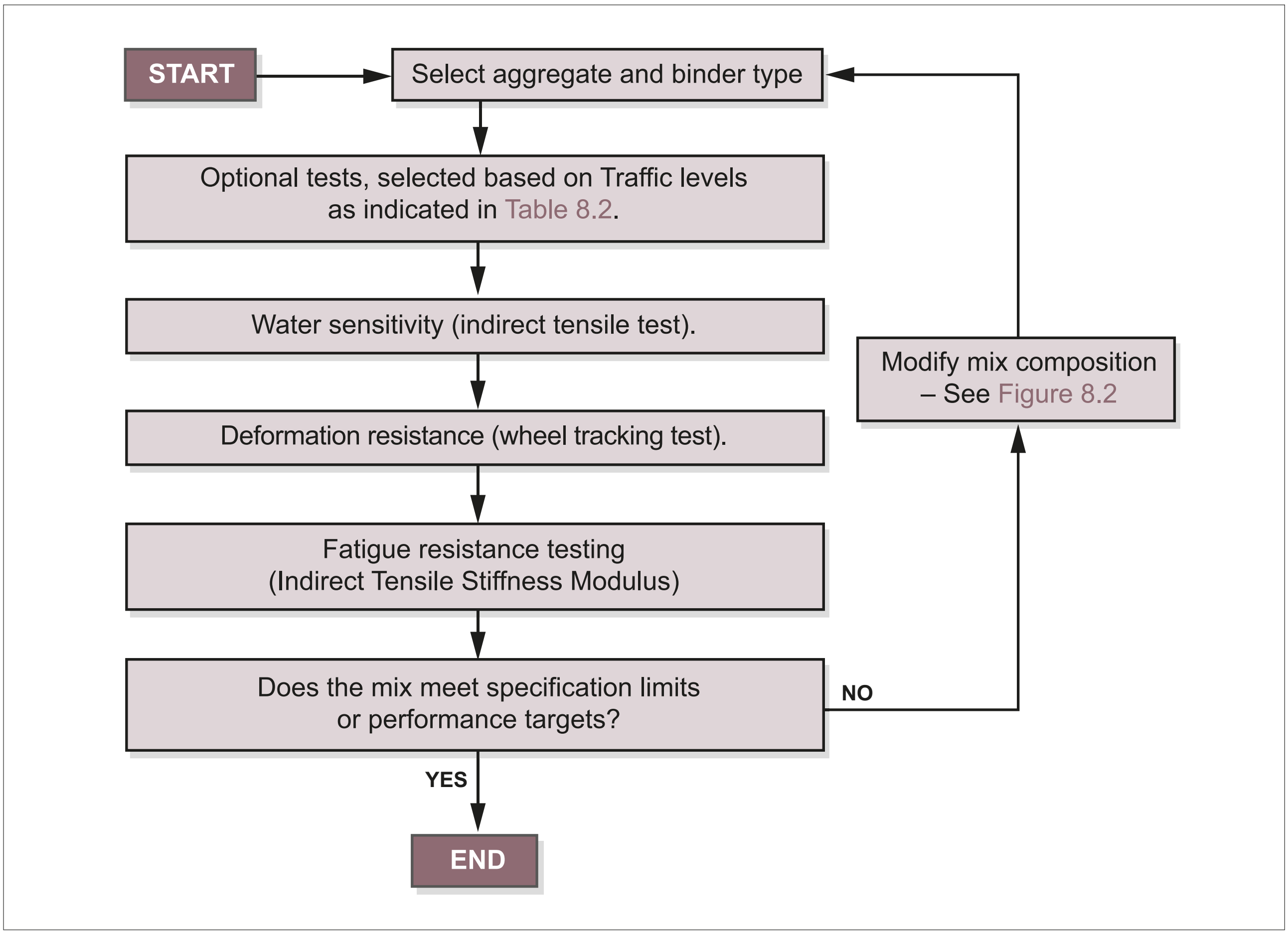


Figure 8.3 Procedure for Performance Testing



8.6.4 Bitumen Stabilised Materials Mix Design Process

The mix design procedure consists of:

1. **Preliminary Tests:**

   These include standard laboratory tests to determine the grading curve, moisture, density relationships and Atterberg limits. Where the results indicate that some form of pre-treatment is required, additional tests must be undertaken after such pre-treatment to ensure that the desired results are achieved.

2. **Level 1 Mix Design:**

   Level 1 starts with the preparation of samples that will be used to manufacture the specimens required for all levels of mix design testing. 100 mm diameter specimens (Marshall briquette) are compacted and cured for Indirect Tensile Strength (ITS) testing. Test results are used to identify the preferred bitumen stabilising agent, determine the optimum bitumen content and to identify the need for filler, and, where required, the type and content of filler.

   Level 1 mix design is sufficient for lightly trafficked pavements, which will carry less than 3 MCESA (BSM50 and BSM100) - see materials Charts BM1 and BM2.

3. **Level 2 Mix Design:**

   This level uses 150 mm diameter by 127 mm high specimens (Proctor specimens) manufactured using vibratory compaction, cured at the equilibrium moisture content, and tested for Indirect Tensile Strength to optimise the required bitumen content.

   This level is recommended for roads carrying more than 3 MCESA (BSM100).

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Pavement Materials

4. **Level 3 Mix Design:**

   This level uses triaxial testing on 150 mm diameter by 300 mm high specimens for a higher level of confidence.

   This step is recommended for design traffic exceeding 10 MCESA (BSM175).

   If a Level 3 mix design is performed, it is not necessary to also do a Level 2 mix design.

   BSM3 based on Level 1 mix design with adjusted material specifications is recommended for LVSR base under this Manual.

The construction of BSMs includes in-situ recycling with recyclers, conventional construction equipment, and in plant treatment. While Foamed Bitumen Stabilised Material (FBSM) requires specialised plant to produce, alternatively with the availability of bitumen emulsion production plant, Bitumen Emulsion Stabilised Material (BSM) can be produced using conventional equipment. BSM50 using bitumen emulsion is therefore recommended for the LVSR under this Manual.

The details of specifications for BSM50 are given in Chart BM1. The mix design procedures shall be provided in the specifications.

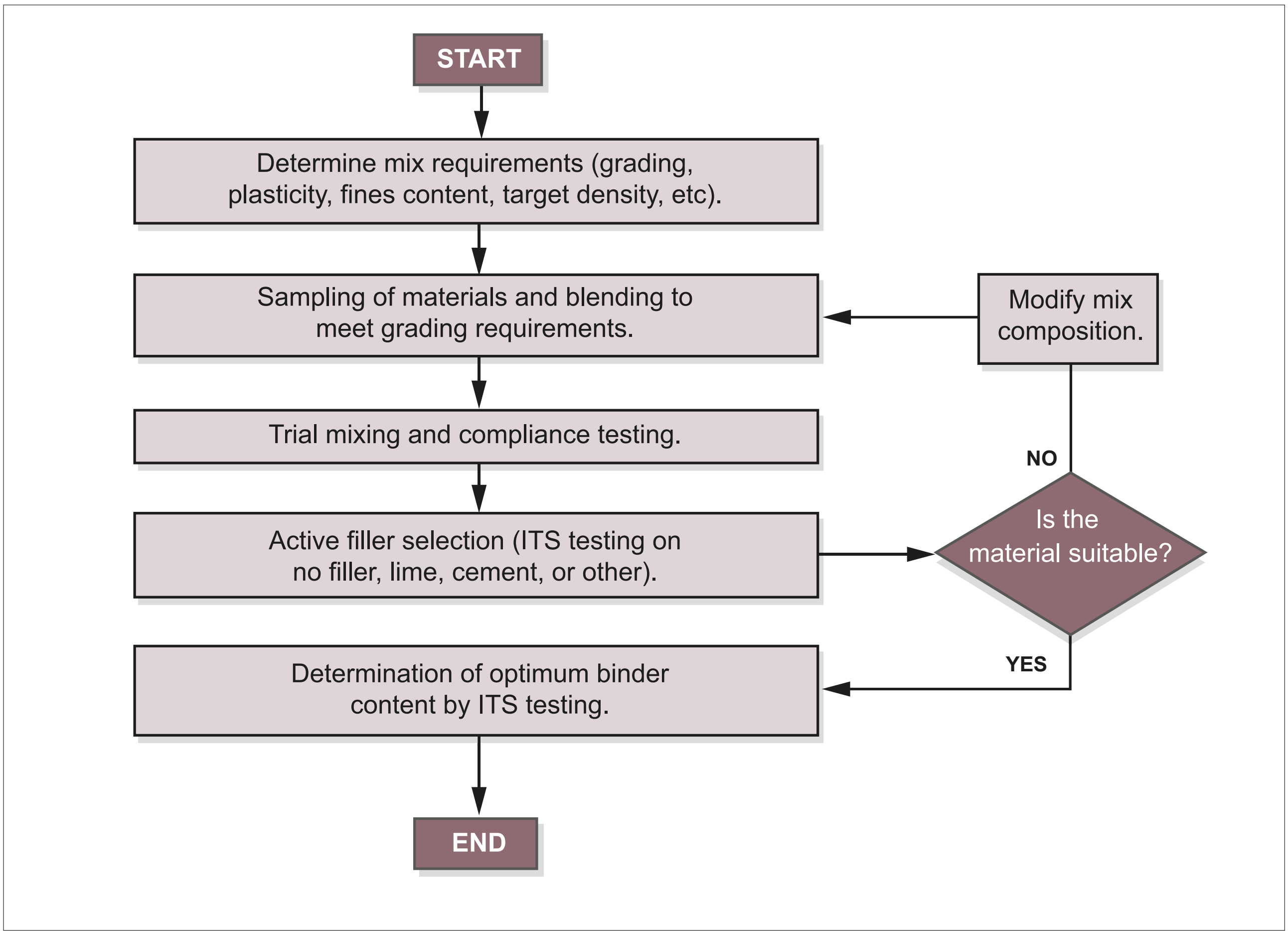
Bitumen emulsion grade A4-60 (anionic grade emulsion of 60 % bitumen content) is the recommended binder for the production of BSM, or if the BSM is to be made using GCS or highly granular material, then K3-60 (cationic grade emulsion of 60 % bitumen content) should be used. Other bituminous binders may also be used provided they can be demonstrated to be workable during construction. The minimum residual binder content acceptable shall not be less than 1.0% by mass. It must be noted that while the bitumen is intended to only coat the fines within the material to facilitate dispersion of the bitumen, the treated material still acts as a granular material.

Natural materials with a PI above 12 % should be mechanically stabilised using suitable materials such as sand, quarry dust or gravel before consideration for pre-treatment with lime, prior to bitumen emulsion stabilisation.

The blended material shall be tested and classified on the basis of Indirect Tensile Strength (ITS). Test specimens of 100 mm diameter are cured until they reach a constant (dry) mass, typically with moisture contents of less than 0.5 %. Testing follows 7 days of curing at 40 ºC, without sealing the specimens to determine the ITSdry value. Half the specimens are then soaked for 7 days before testing to determine the ITSwet value. This procedure is aimed at evaluating the moisture susceptibility of the BSM.

A flow chart summarising the mix design procedure is as shown in Figure 8.4.

Figure 8.4 Mix Design Procedure for BSM



8.6.5 EME (High Modulus Asphalt) Mix Design Process

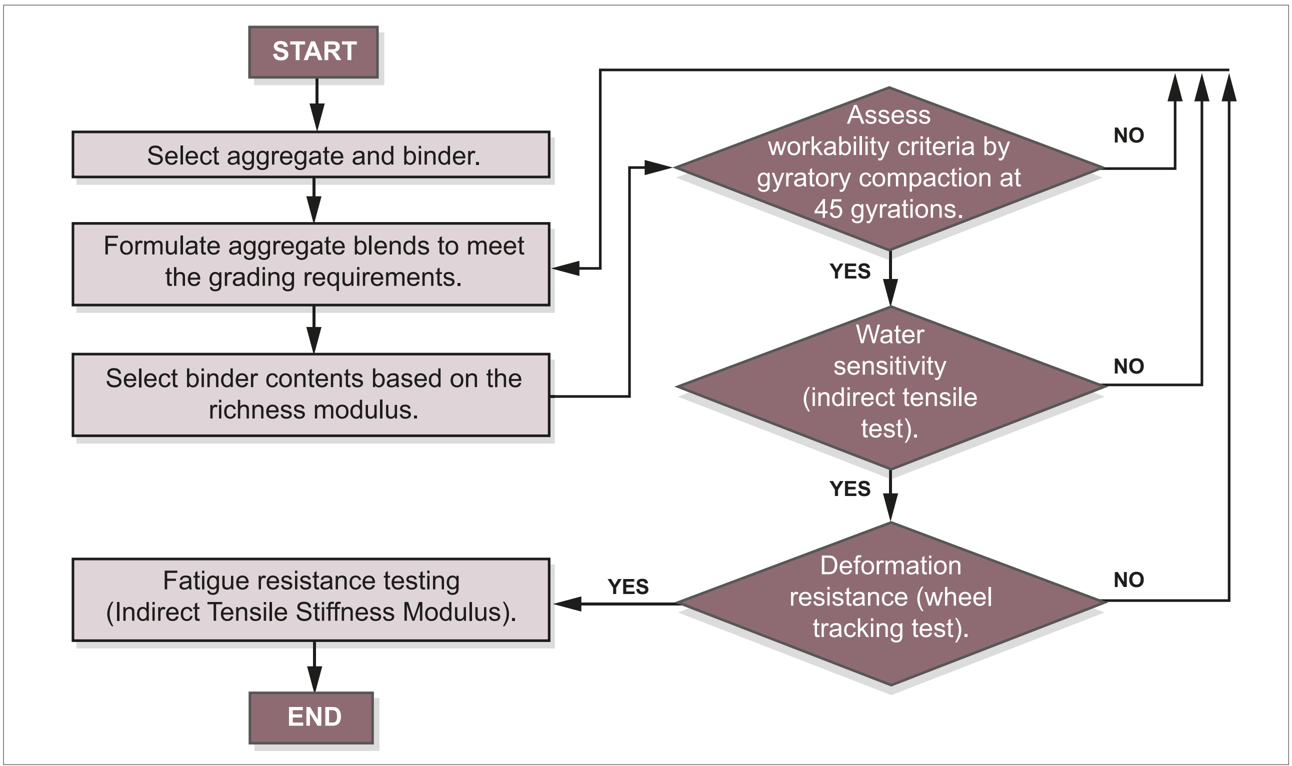
The design process is as follows:

1. Select appropriate mix components in terms of aggregate and binder.
2. A suitable grading is developed from the different aggregate fractions.
3. The binder content is determined based on a minimum richness modulus, similar to the film thickness conventionally used in SA.
4. Using a trial mix design, specimens are compacted in a gyratory compactor. A maximum allowable air void content after a set number of gyrations has to be met. This is the first of the performance criteria, aimed at creating a workable mix.
5. Once workability criteria have been met specimens are subjected to a durability test.
6. Following satisfactory durability, the following structural performance criteria are assessed:
    a. Minimum dynamic modulus.
    b. Minimum level of resistance to permanent deformation.
    c. Minimum fatigue life.

Specifications are given in Chart BB2.

A flow chart summarising the mix design procedure is as shown in Figure 8.5.

Figure 8.5 Mix Design Procedure for EME (High Modulus Asphalt)



8.6.6 Stone Mastic Asphalt Design Process

A critical aspect of SMA is to form a mastic (bitumen + fine aggregate + filler) that is voidless prior to compaction, but in the presence of larger stone particles, the compacted air voids should be 3 % – 5 %.

The design of SMA consists of 3 main aspects:

1. Design of the stone skeleton.
2. Design of the mastic.
3. Design of the mix.

Design of the Stone Skeleton

1. The grading selected for a 14 mm nominal maximum aggregate size (NMAS) should have a pronounce gap between 0.5 and 5 mm.
2. Determination of the volume of air in between the coarse aggregate particles when subjected to dry rodding in accordance with AASHTO T19.

Design of the Mastic

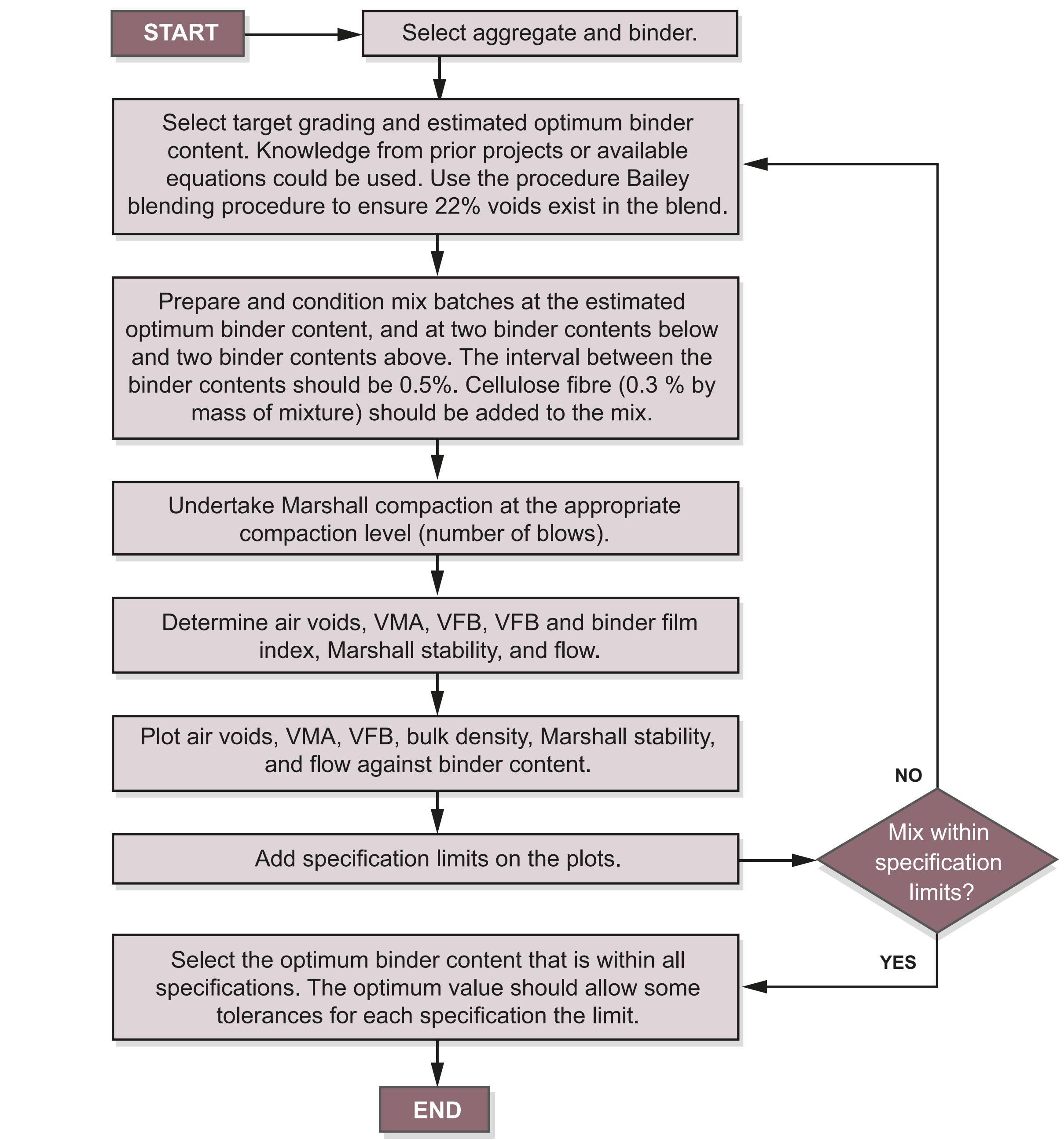
1. The fine aggregate should be selected such that all particles should be between 0.075 mm and 2 mm.
2. Filler of less than 0.075 mm should be used.
3. The ratio of aggregate to filler should be 4:1 or higher by mass.

Design of the Mix

1. For a selected fine aggregate: filler ratio and bitumen content, the proportion of coarse aggregate (2-14 mm) is varied e.g. 60 %, 75 %, and 80 %.
2. The overall voids of the compacted mix (Vibratory Hammer or Marshall). The target is between 3 % − 5 %.
3. A check to ensure that the voids (comprising the mastic + air) in between the coarse aggregates after compaction of the mix is less than rodded voids determined by AASHTO T19 earlier in the design process.

A flow chart summarising the mix design procedure is as shown in Figure 8.6. This approach is also applicable to the design of other variations of gap-graded mixes (e.g., SMA without fibre). Specifications are given in Chart SU11.

Figure 8.6 Mix Design Procedure for Stone Mastic Asphalt



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8.7 Surface Dressing Design

The design of surface dressing involves the following steps:

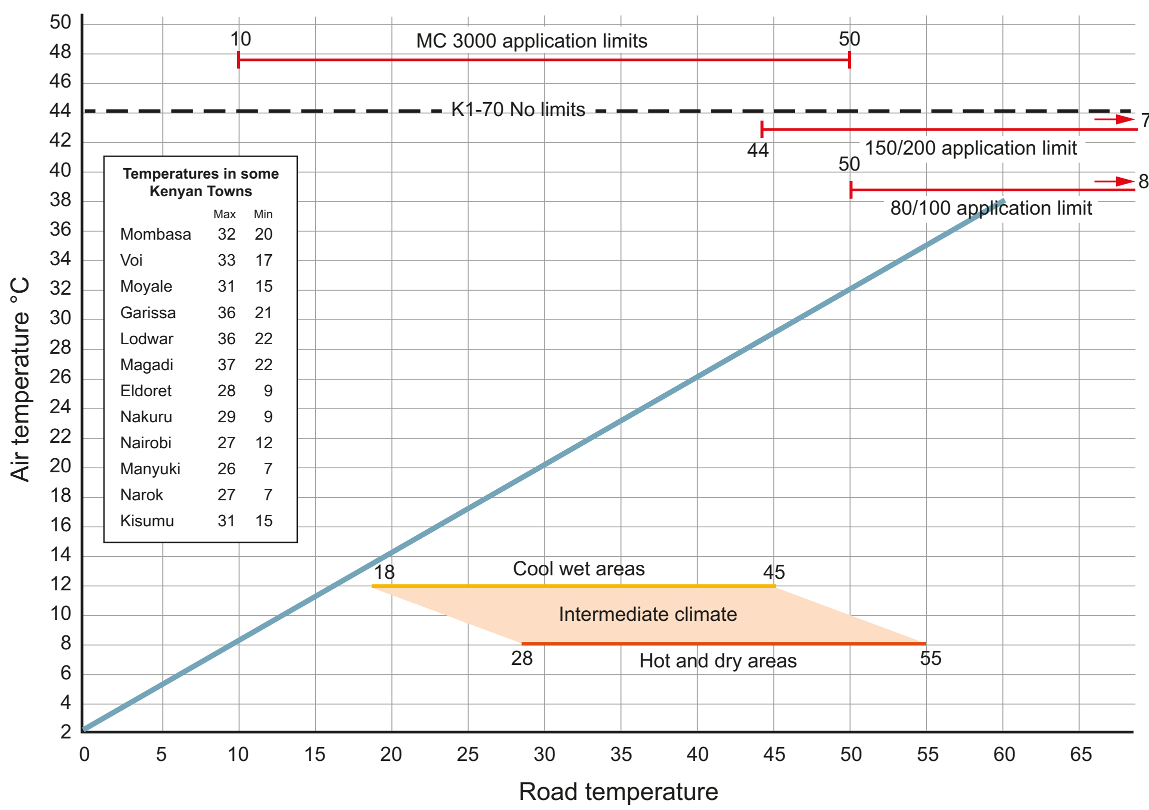
1. Estimate the road surface temperature from Figure 8.7.
2. Select the appropriate bitumen grade from Figure 8.8.
3. Determine the Average Least Dimension (A.L.D.) of the chippings.
4. The determination of the chipping application rates is based on the Average Least Dimension (A.L.D.) of the chippings. It is recommended that the A.L.D. be determined by direct measurements rather than by estimating it from the median size and the Flakiness Index of the chippings. The application rate of binder is based on the chipping application rate and read-off from a chart (Figure 8.9).
5. Using the chippings application rate, determine the residual binder application rate from the multipliers in Figure 8.9.

The chippings spread rates obtained from Figure 8.7 strictly apply to the lower seal(s) of multiple surface dressings. For single surface dressings or the top seals of multiple surface dressings these rates should normally be increased by about 10 %, to achieve a good coverage.

As regards the application of emulsion, K1-70 would run off the road if its rate exceeded about 1.2 l/m^2 on a smooth primed base and 1.5 l/m^2 on a chipping-seal. Consequently, with emulsion, the application rates should be calculated as follows:

a. Determine the total amount of residual bitumen required.
b. Calculate the total amount of emulsion required.
c. Split this amount into 2 sprays for single surface dressing and 3 sprays for double surface dressing, so that no run-off will occur, and the upper spray rate is minimsed. (Because of its low viscosity, emulsion flows down and fills the voids between chippings).

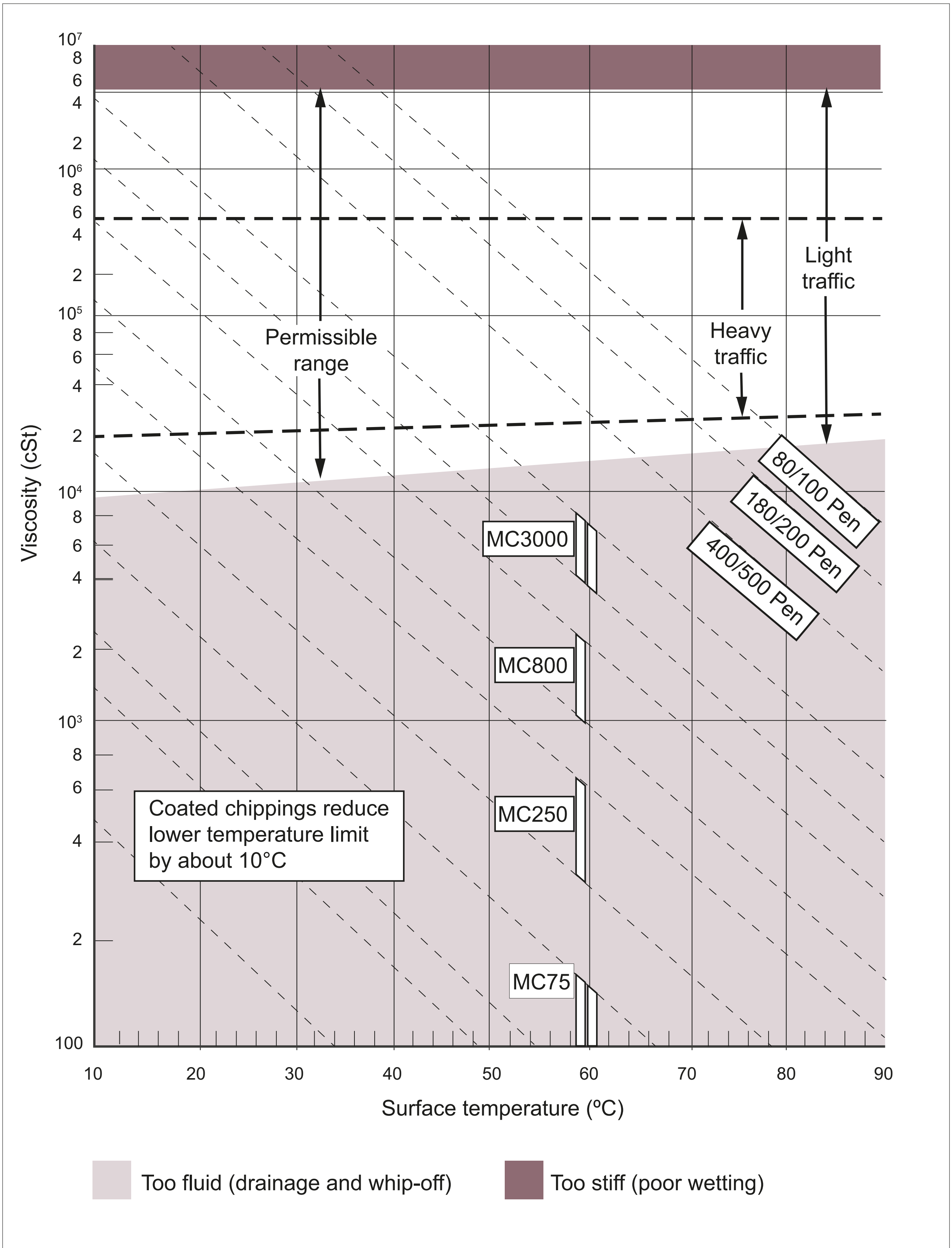
Figure 8.7 Road Temperature from Air Temperature



Temperatures in some Kenyan Towns

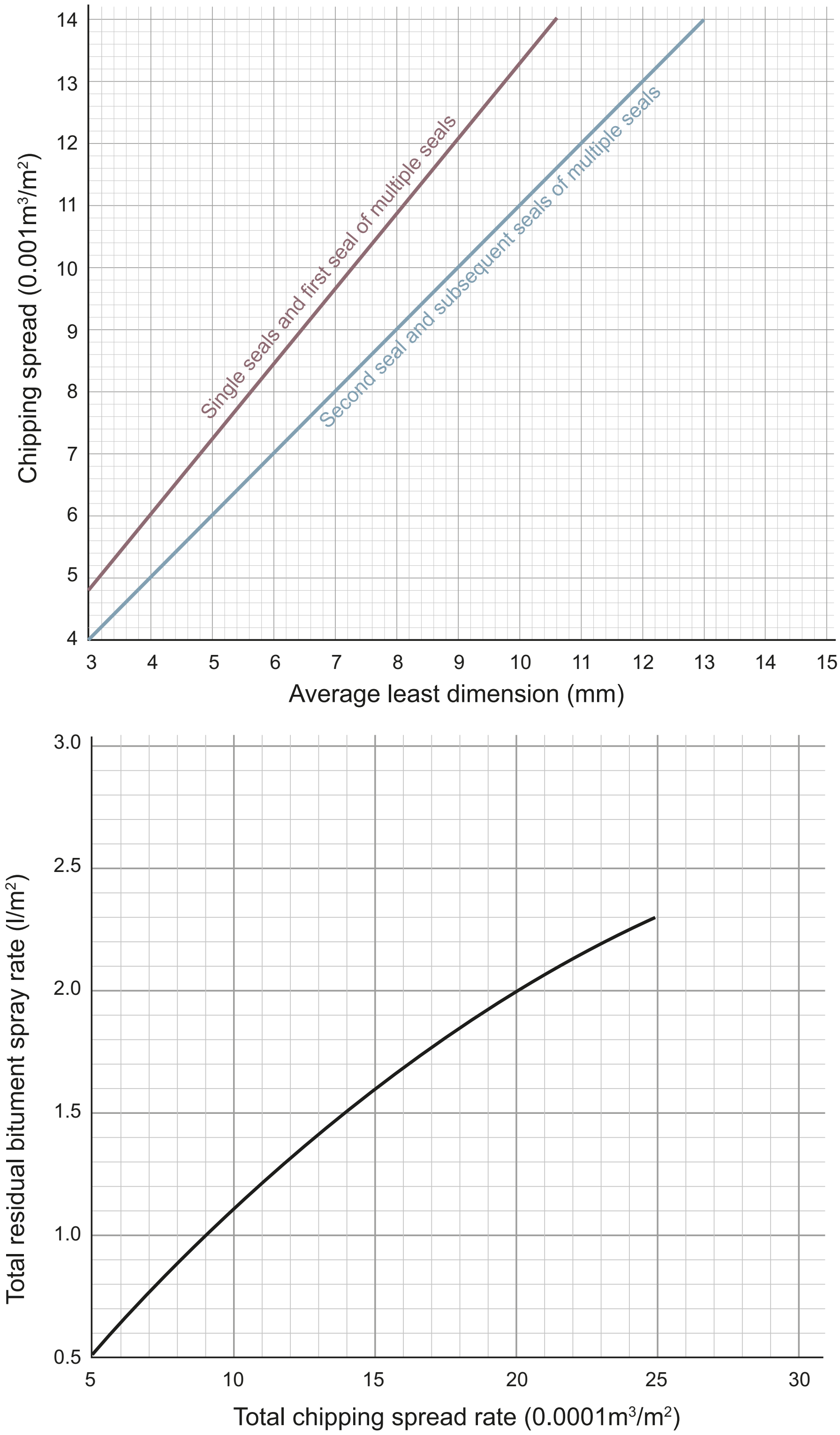
	Max	Min
Mombasa	32	20
Voi	33	17
Moyale	31	15
Garissa	36	21
Lodwar	36	22
Magadi	37	22
Eldoret	28	9
Nakuru	29	9
Nairobi	27	12
Manyuki	26	7
Narok	27	7
Kisumu	31	15

Figure 8.8 Viscosity-road Temperature Relationship



Note: *For precoated chippings, surface temperature should be shifted by 10 C lower before selecting a suitable bitumen grade.*

Figure 8.9 Chipping Spread Rate and Bitumen Rate Multipliers



Traffic v.p.d	Multiplier	Climate	Multiplier	Surface Texture	Multiplier	Chippings	Multiplier
0 – 100	1.13	N. European	1.10	Primed base	1.00	Round/dusty	1.05
100 – 500	1.00	Tropical	1.06	V. lean bituminous	0.95	Cubical	1.00
500 – 1000	0.96	Temperate	1.03	Lean bituminous	0.85	Flaky (FI<35)	0.95
1000 – 2000	0.90	Semi-Arid	1.00	Average bituminous	0.80	precoated	0.95
> 2000	0.85	Arid	0.95	V. rich bituminous	0.75		

8.8 Concrete

The Constituents of the concrete mix shall not contain harmful ingredients in such quantities as may be detrimental to the durability of the concrete or cause corrosion of the reinforcement and shall be suitable for the intended use in concrete.

Where possible locally available materials should be used for economic and environmental benefits. This will also reduce damage to the road network.

8.8.1 More Environmentally Friendly Concrete (Geopolymer Concrete)

Designers should be aware that the production of Ordinary Portland Cement (OPC) involves large CO_2 emissions, which can contribute to global warming.

More environmentally friendly concretes are available such as Geopolymer concrete, also known as Earth friendly concrete (EFC), as well as conventional cements and concretes. This concrete can reduce CO_2 emissions by 80 % compared to Portland cement. Other benefits include high flexural strength, low shrinkage and good workability making it very suitable for use as a pavement concrete.

Instead of ordinary Portland cement, it contains a combination of by-products from industrial processes such as water purification, waste incineration and steel production, along with quarry dust, agricultural waste, and recycled aggregates. The formation of geopolymers is based on aluminosilicate, which hardens when an alkaline activator is added (usually a combination of sodium silicate solution (water glass) and sodium hydroxide (NaOH)).

The use of geopolymer concrete is still in the development stage worldwide, although it has been rapidly advancing in Europe (not UK) and Australia.

8.8.2 Cement

Cement to be used for a road pavement shall comply with KS EAS 18-1 (Composition, specification & conformity criteria for common cements) and should be tested in accordance with KS EAS 148-1 (Cement-Test methods).

They comprise finely ground powder of hydraulic binders or mixtures of hydraulic binders and non-hydraulic binders. A hydraulic binder is a powder that reacts with water by hydration to form pastes of insoluble hydrates which sets and hardens with time even under water.

Portland cement is the most common hydraulic binder and is formed by grinding Portland cement clinker which is manufactured as follows:

a. Crushing and grinding of raw material consisting of calcareous material such limestone, required provide lime (CaO), which makes approximately 75-85% of the raw material mix and shale, clays, iron ore, bauxite and sand which provide the oxides of silica, aluminium and iron.

b. Blending the materials in correct proportions.

c. Burning the mix in a kiln to produce ‘clinker’.

d. Grinding the clinker (with approximately 5 % gypsum to control the setting time) to form a fine powder.

Cement is usually mixed with coarse aggregate, fine aggregate, and water (and sometimes other additives) to form concrete.

As well as the basic Portland cement (known as CEM I), there are many different types of cement, based on combinations of CEM I and other hydraulic binders (e.g. ground granulated blast furnace slag, ggbs), and pozzolanic cements (e.g. fly ash, fa).

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There are five main cement types:

1. CEM I = Portland cement
2. CEM II = Portland composite cement
3. CEM III = Blast furnace cement
4. CEM IV = Pozzolanic cement
5. CEM V = Composite cement

The most common types of combinations of these cements are given in Table 8.3.

For concrete pavements, the following cement types are usually used. The general term ‘cement’ in this Manual means any of the materials in (i) or the combinations in (ii) below:

i. Cements (Complying with KS EAS 18-1):

a.	Portland cement	CEM I
b.	Portland-slag cement	CEM II/A-S and CEM II/B-S
c.	Blastfurnace cement	CEM III/A, III/B and III/C
d.	Portland-fly ash cement	CEM II/A-V and CEM II/B-V
e.	Pozzolanic cement	CEM IV/A

ii. Combinations (Complying with KS EAS 18-1, BS EN 15167-1, -2, BS EN 450-1,-2).

a. Portland cement CEM I with ground granulated blastfurnace slag (ggbs) (as specified in BS EN 15167-1 and BS EN 15167-2).

b. Portland cement CEM I with fly ash (fa) for use as a cementitious component in structural concrete (BS EN 450-1 and BS EN 450-2).

c. Portland cement CEM I with pozzolanic additive (BS 6610).

Blended cements can have properties such as high early strength cement and limited shrinkage cement.

In each cubic metre of fully compacted concrete the cement content shall be in accordance with Table 8.4.

Table 8.3 Common Types of Cement

Main Cement Type	Notation of 27 Types of Common Cements		Main Constituents % by mass (a)										Minor additional constituent % by mass
			Clinker	Blast Furnace	Silica fume	Pozzolana		Fly Ash		Burnt Shale	Limestone		
						Natural	Natural Calcined	Siliceous	Calcareous				
			K	S	D(b)	P	q	V	W	T	L	LL	
CEM I	Portland cement	CEM I	95-100	-	-	-	-	-	-	-	-	-	0-5
CEM II	Portland slag cement	CEM II/A-S	80-94	6-20	-	-	-	-	-	-	-	-	0-5
		CEM II/B-S	65-79	21-35		-	-	-	-	-	-	-	0-5
	Portland silica fume cementt	CEM II/A-D	90-94	-	6-10	-	-	-	-	-	-	-	0-5
	Portland pozzolana cement	CEM II/A-P	80-94	-	-	6-20	-	-	-	-	-	-	0-5
		CEM II/B-P	65-79	-	-	21-35	-	-	-	-	-	-	0-5
		CEM II/A-Q	80-94	-	-	-	6-20	-	-	-	-	-	0-5
		CEM II/B-Q	65-79	-	-	-	21-35	-	-	-	-	-	0-5
	Portland fly ash cement	CEM II/A-V	80-94	-	-	-	-	6-20	-	-	-	-	0-5
		CEM II/B-V	65-79	-	-	-		21-35	-	-	-	-	0-5
		CEM II/A-W	80-94	-	-	-	-	-	6-20	-	-	-	0-5
		CEM II/B-W	65-79	-	-	-	-	-	21-35	-	-	-	0-5
	Portland burnt shale cement	CEM II/A-T	80-94	-	-	-	-	-	-	6-20	-	-	0-5
		CEM II/B-T	65-79	-	-	-	-	-	-	21-35	-	-	0-5
	Portland limestone cement	CEM II/A-L	80-94	-	-	-	-	-	-	-	6-20	-	0-5
		CEM II/B-L	65-79	-	-	-	-	-	-		21-35	-	0-5
		CEM II/A-LL	80-94	-	-	-	-	-	-	-	-	6-20	0-5
		CEM II/B-LL	65-79	-	-	-	-	-	-	-	-	21-35	0-5
	Portland composite cement (c)	CEM II/A-M	80-88	12-20									0-5
		CEM II/B-M	65-79	21-35									0-5
CEM III	Blastfurnace cement	CEM III/A	35-64	36-65	-	-	-	-	-	-	-	-	0-5
		CEM III/B	20-34	66-80	-	-	-	-	-	-	-	-	0-5
		CEM III/C	5-19	81-95	-	-	-	-	-	-	-	-	0-5
CEM IV	Pozzolanic cement (c)	CEM IV/A	65-89	-	11-35					-	-	-	0-5
		CEM IV/B	45-64	-	36-55					-	-	-	0-5
CEM V	Composite cement (c)	CEM V/A	40-64	18-30	-	18-30			-	-	-	-	0-5
		CEM V/B	20-38	31-49	-	31-49			-	-	-	-	0-5

Notes: ***(a)*** *Values in the table refer to the sum of the main and minor constituents (percentage by mass).*
(b) *The proportion of silica fume is limited to 10 %.*
(c) *In Portland composite cements CEMII/A-M and CEM II/B-M, in Pozzolanic cements CEM IV/A and IV/B and in composite cements CEM V/A and V/B the main constituents other than clinker shall be declared by designation of the cement.*

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Table 8.4 Minimum Cement (or Combination) for 40 mm Max Aggregate

Class BS 8500-1 Cement	C40/50	C32/40 In slabs over-laid by > 30 mm asphalt	C32/40 In at least top 50 mm of slabs	C25/30	C16/20	C12/15	C8/10	C6/8
Minimum Portland Cement CEM1 (KS EAS 18-1) (kg/m^3)	BS8500	320	320	280	180	160	130	120
Minimum other cements or combinations permitted in (i) & (ii) above (kg/m^3)	–	340	340	300	180	160	130	120
Mixtures pre-blended or mixed on site								
Maximum proportion of ggbs (%)	50	50	35	65	65	65	65	65
Max/Min. proportions of fly ash (%)	35/15	35/15	25/15	35/15	35/15	50/0	50/0	50/0
Minimum CEM1 content (kg/m^3)	220	220	255	200	160	–	–	–

***ggbs** = granulated blast furnace slag, **fa** = fly ash, **pfa** = pulverised fly ash*
***Note:** For 20 mm maximum size aggregate add 20 kg/m^3, and for <20mm maximum size add 40 kg/m^3.*

Portland limestone cement should not be used in the top 50 mm of the road surface, as this would increase the fine calcium carbonate content and lead to slipperiness. Microsilica may be used with CEM I to obtain high early strength concrete.

For durability it is necessary to have a water/cement content ratio below 0.45 for pavement surface slabs. The water/cement ratio is defined as the ratio of free water to total cementitious content of the concrete.

High early strength cements, high cement content and a low water/cement ratios may be used when there is a need to open a section of concrete pavement to traffic early. Prescribed concretes of fixed proportions may be used in rapid construction for high early strength concrete. The proportions of ingredients to be used should be decided by trial concrete mixes which when tested provide the quality, consistence and strength development required for the particular application.

Both CEM I /fa and CEM I /ggbs concrete have a long-term increase in strength greater than CEM I concrete for the same 28-day strength and provide greater durability and resistance to chemical attack. If fa is included in the concrete, it permits lower water/cement ratios for a required consistence, so providing denser concrete of lower permeability and greater durability.

8.8.3 Coarse Aggregate

Aggregates shall comply with KS EAS 131 (Concrete - Part 1: Specification, performance, production and conformity.), KS 95 (Specification for natural aggregates for use in concrete), and KS 1238 Parts 1-20 (Methods of tests for aggregates).

The maximum aggregate size should not exceed one third (1/3) of the slab thickness. The maximum aggregate size allowed is 40 mm (less for RCC), but the Contractor’s choice of size will depend on construction methods, and his ability to achieve surface regularity, properly constructed joints, and correct alignment of dowels.

Larger aggregate provides more stable concrete in the lower layer, but smaller aggregate (max. 20 mm) is preferable in the top course for forming joints and achieving a good finish. Some engineers choose aggregates to be less than 20 mm to minimise larger aggregate sinking to the bottom of the mix when being spread and compacted (which can weaken the concrete).

For JRC or CRCB/CRCP pavements the maximum aggregate size shall not exceed on third (1/3) of the spacing between longitudinal steel bars. If the spacing is less than 90 mm, the maximum size of coarse aggregate (D) shall not exceed 20 mm.

The Contractor shall submit the declaration of performance for each aggregate to the Overseeing Organisation prior to the incorporation of the aggregate into the works. The declaration of performance shall demonstrate that the aggregate meets the specification requirements.

The properties of aggregates suitable for use in a concrete mix vary according to:

a. The natural properties of the material e.g., toughness, durability, soundness, density, water absorption/porosity, surface microtexture and chemical properties e.g., alkali reactivity and thermal expansion.

b. The properties that can be controlled e.g., shape, size, distribution, and cleanliness.

   There are numerous tests for the suitability of aggregates in concrete mixes which include:

   1. Resistance to Alkali-Silica Reaction (ASR).
   2. Resistance to fragmentation for concrete surface slabs.
   3. Flakiness Index.
   4. Abrasion Resistance.
   5. Chloride Content.
   6. Water Absorption (WA) of the coarse aggregate.
   7. Chemical requirements (Acid-soluble sulphate content of the aggregates and total sulphur content of recycled aggregates).
   8. Resistance to freeze/thaw - does **NOT** apply to Kenya.

Aggregate with a relatively high bond strength with the mortar, such as limestone, usually exhibit less spalling than a siliceous gravel aggregate.

In concrete, it is beneficial to use aggregates with a low coefficient of thermal expansion (e.g. limestone) so that slab movement, and hence problems at joints, are minimised. Limestone, however, can be polished by traffic, leading to dangerously slippery road surfaces in the wet. In wet climates, it can be prudent to limit limestone aggregate concrete to more lightly used roads or use a two-layer approach to building the concrete slab, with non-limestone aggregate in the upper (surfacing) layer.

Coarse aggregates shall consist of clean, hard, strong, non-porous pieces of crushed stone or crushed gravel. Continuously graded, or gap-graded, aggregates may be used, depending on the grading of the fine aggregate.

Aggregates for all pavement concrete, including Lower Strength, shall comply with the relevant standard. Coarse aggregate may be crushed air-cooled blast-furnace slag Category FI50 or FI35 for concrete of classes C16/20 to C25/30 and over class C25/30, respectively. Once the appropriate gradings have been determined they shall not be varied without the approval of the Overseeing Organisation.

Reclaimed Aggregate

Reclaimed aggregate may be used as aggregate for all pavement quality concrete (apart from RCC) provided that:

i. It shall not be added in quantities greater than 5 % by mass of the total aggregate if they are undivided,

ii. Where the quantities of the reclaimed washed aggregates are greater than 5 % by mass of the total aggregate, they shall be divided into separate coarse and fine aggregates and conform to the relevant standards for recycled aggregates.

Crushed concrete, which complies with the quality and grading requirements of the relevant standard, may also be used in all pavement concretes except exposed aggregate concrete surfacings.

8.8.4 Fine Aggregate

As with coarse aggregates, fine aggregates shall comply with KS EAS 131 (Concrete - Part 1: Specification, performance, production and conformity.), KS 95 (Specification for natural aggregates for use in concrete), and KS 1238 Parts 1-20 (Methods of tests for aggregates).It should be noted that KS EAS 131 still references BS EN 12620+A1 (Aggregates for concrete), whereas it should probably refer to KS 1238 (Specification for road aggregates and Methods of tests for aggregates).

Where natural sand is used as the fine aggregate, there is a temptation to use sand at the finer end of the grading, as it is often cheap and plentiful and cheaper, but this will produce concrete with poor workability, which can be particularly problematic in hotter climates, with more rapid hardening times. It is better to use sand with gradings near the coarse limits.

8.8.5 Water

KS 2846 (Mixing water for concrete) should be used as the standard. It should be noted that KS EAS 131-1 (Concrete Part 1: Specification, performance, production, and conformity) is still current but states that water for mixing in concrete shall conform to BS EN1008.

Water to be used for concrete should generally be free from oils, acids, alkalis, and organic impurities. Generally water that is fit for drinking can be used for mixing and curing concrete.

However, when working in remote areas it may not be possible to get potable water and the project may have to use water that is not potable. The following should be noted:

1. Soft water produces weaker concrete than hard water.
    - Soft water has hardness as $CaCO_3$ = less than 50mg/l,
    - reasonably soft = 50 to 100 mg/l,
    - Slightly hard = 100 to 150 mg/l,
    - Reasonably hard = 150 to 250 mg/l,
    - Hard = 250 to 350 mg/l,
    - Very hard = more than 350mg/l.
    - Rainwater is usually soft as it has very little opportunity to absorb chemicals.
2. Water from marshes is generally not suitable for concrete.
3. Water containing decaying vegetation is undesired as this contamination interferes with cement setting.
4. Salt water should generally NOT be used for any concrete works as it will corrode tie bars, dowel bars and steel reinforcement. Sea water will also increase setting times and reduce the long-term strength by up to 15 % (Neville, 2011) and increase the risk of efflorescence.

8.8.6 Admixtures

Standards for admixtures are given in KS 2770 (Admixtures for concrete) and KS 2769 (Admixtures for concrete - Test methods).

It should be noted that KS EAS 131-1 (Concrete Part 1: Specification, performance, production, and conformity) states that Admixtures and any other additive materials should conform to BS EN 934-2.

Admixtures are optional and may be added to the cement or concrete mix by the designer to enhance the properties of the concrete. Each type of admixture serves a different purpose and is used under different circumstances. For example, they can improve the workability of the concrete, extend the setting time, etc.

The most common types of admixtures include:

1. **Retarders.** These are the most common type of admixtures. They increase the setting time of the concrete and can improve the workability /appearance of the concrete.
2. **Pozzolans** (e.g., crushed rock powder, bentonite, fly ash and ground granulated blast furnace slag). These generally 'improve' concrete by converting Calcium Hydroxide (CH) (a by-product of the hydraulic reaction between water and cement powder) to additional Calcium Silicate Hydrate (CSH) which is the glue that binds the concrete aggregate. They are often added to Ordinary Portland Cement, to control setting, increase durability, reduce cost and/or reduce pollution without compromising the compressive strength. Plasticisers are also often used when pozzolanic ash is added to concrete to improve strength.
3. **Plasticisers** (these can reduce water content in the mix, increase strength and maintain consistency) Also beneficial in cements with added ggbs or pfa as the water reduction can compensate for some of the early strength loss. Plasticisers or water reducing admixtures shall comply with the relevant standard. Admixtures containing calcium chloride shall not be used.
4. **Superplasticisers (Water-Reducing Admixtures)** (e.g., lignosulfonates, hydro-carboxylic acids, polycarboxylates and sulfonated naphthalene formaldehyde). They are added to reduce the amount of water needed for mixing and placing of concrete. They can improve the workability, strength, durability, and shrinkage resistance of concrete by lowering the water to cement ratio. They work by creating an electrostatic repulsion between cement particles, stopping them clumping together and allowing them to disperse more evenly in the water. This reduces the amount of water required to achieve a certain level of consistency or slump.
5. **Air-entraining admixtures.** The main application of this is to reduce freeze/thaw damage, so it is not required for this purpose in Kenya. (Note that a C40/50 or stronger concrete, or a C32/40 concrete to be overlaid by at least 30 mm asphalt, is less susceptible to this damage anyway). There may be occasions where air entrainment is used to increase workability, but it should be noted that this can reduce its strength.
6. **Shrinkage Reducing Admixtures (SRAs)**. These do not affect the mechanical properties of the concrete, but they considerably reduce the shrinkage potential and shrinkage rate of the concrete mix. It should be noted that the initial and final set may be delayed, and the concrete may bleed for slightly longer.
7. **Other types of admixtures include:**
    - Accelerators (to speed up the setting process)
    - Corrosion inhibitors (to protect the steel bars/reinforcement)
    - Alkali aggregate expansion inhibitors and colour dyes.

The total amount of admixtures, if any, shall not exceed the maximum dosage recommended by the admixture producer and shall not exceed 50 g of admixture (as supplied) per kilogram of cement unless the influence of the higher dosage on the performance and the durability of the concrete is established and taken into account. [KS EAS 131-1, Para 5.2.6]

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Admixtures used in quantities less than 2 g/kg of cement shall be dispersed in part of the mixing water except where the admixture cannot be dispersed homogeneously in the mixing water (e.g. because it forms a gel). In this case, other methods of feeding into the concrete may be used.

If the total quantity of liquid admixtures exceeds 3 l/m^3 of concrete, its water content shall be taken into account when calculating the water/cement ratio. [KS EAS 131-1, Para 5.2.6]

Where more than one admixture is used, the compatibility of the admixtures shall be checked in the initial tests. The use of any chemicals to retard the curing process must be properly tested to ensure their effectiveness and whether they have any damaging effects on the concrete.

8.8.7 Air Content

The maximum air content (as a percentage of the volume of the concrete) shall be as given in Table 8.5.

Table 8.5 Maximum Air Content

Maximum Aggregate Size (mm)	**Maximum Air Content** (% volume)
20	6.5
40	5.5

The air content shall be determined at the point of delivery to the paving plant by the pressure gauge method at the rate of one determination per 300 m² of slab or at least 6 times per day, whichever is the greater, in conjunction with tests for consistence and strength. For areas less than 300 m², the rate shall be at least one determination to each 20 m length of slab or less constructed at any one time or at least 3 times per day. If the air content exceeds the maximum specified limit in the required specification, then the Contractor shall remove the concrete from the works.

Air entrainment is usually used to increase the durability of the concrete, particularly to protect it from freeze/thaw damage which can result in surface scaling. The millions of microbubbles (each 0.1-1.0mm in diameter) relieve the pressure as water in capillary cracks freezes and expands.

It should be noted that each 1 % of entrained air reduces the compressive strength of the concrete by approximately 3-6 %. The usual amount of air entrainment is 3-6 % for durability, which equates to a compressive strength reduction of 9-36 %.

If an air-entrained concrete is used, then the air-entraining agent shall be added at the mixer, by an apparatus capable of dispensing the correct dose within the tolerance for admixtures given in the specification, to ensure uniform distribution of the agent throughout the batch during mixing. It should be noted that if transported long distances the amount of air in the concrete will reduce and this should be taken into account.

8.8.8 Concrete Temperature

KS EAS 131-1 (Concrete - Part 1: Specification, performance, production & conformity) states that the temperature of the fresh concrete shall not be less that 5 °C at the time of delivery.

Where a requirement for a different minimum temperature or a maximum temperature of fresh concrete is necessary, they shall be specified also giving tolerances. Any requirement for artificial cooling or heating of the concrete prior to delivery has to be agreed between the producer and the user.

8.8.9 Fibre Reinforced Concrete

Currently there does not appear to be a relevant KS or EAS standard for fibres, hence general suitability for fibres is established the following standards:

- **Steel fibres conforming to BS EN 14889-1.**
- **Polymer fibres conforming to BS EN 14889-2.**

Fibre reinforced concrete (FRC) (or specifically Steel fibre reinforced concrete (SFRC)) is concrete containing short, discrete fibres that are uniformly distributed and randomly orientated. The fibres are usually steel but glass, carbon, fibre, other synthetic materials (e.g., polyester or polypropylene) and organic materials have also been used. Polymeric fibres are reported to be increasingly used for their cost effectiveness and the fact that they don't corrode when exposed to salt water, etc.

Fibres are usually used in concrete to make it more crack resistant and to bind the cracks together if it does crack. The addition of fibres will have little effect on the concrete's compressive strength but does increase the flexural strength. For heavily trafficked roads, fibres should not replace the steel bar reinforcement. FRC and SFRC are more widely used in thin overlay applications than in new construction.

The most widely used fibres are steel and come in variety of shapes designed to improve their anchorage in the concrete (e.g., wavy, crimped end or enlarged end). Steel fibres are typically 20 - 100 mm long (with 60 mm being the most common length) and a diameter of at least 50 mm.

The steel fibres are distributed into the concrete mix before laying – it can be difficult to get an even distribution. Fibre length is very important - if they are too long then they tend to 'ball up' and interfere with the workability of the concrete. An uneven distribution of fibres can have a negative effect on the concrete strength.

Many sources say that fibres should NOT normally be used with conventional steel bar reinforcement as the issue of reduced workability of the concrete can lead to poor compaction beneath the reinforcement.

It should be noted that steel fibres added to FRC are often classed as a concrete enhancement. SFRC can be expensive, and it is generally not economic to replace the steel reinforcement in CRCB/CRCP with steel fibres. Steel fibres with zinc coatings should not be used unless it is shown that hydrogen formation in the concrete is prevented.

Specialist knowledge is required for the use of FRC. The number of fibres added to a concrete mix is expressed as a percentage of the total volume of the composite (concrete and fibres), termed 'volume fraction' (Vf), with typical values ranging from 0.1 to 3 %. Distribution rates should be as recommended by the manufacturer. Often, when using fibres, the concrete mix will need to be adjusted e.g., the aggregate grading, maximum aggregate size may need to be changed and superplasticisers will probably also be required. It is recommended that shrinkage reducing admixtures are used in all FRC and SFRC.

The finished surface of the pavement should also be considered. If it is to remain a trafficked concrete surface, then there are the issues of a) sharp steel fibres being exposed with possible risks to causing punctures and injuring people walking on the road and b) rusted exposed steel fibres staining the surface and being unsightly.

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8.8.10 Workability

For concrete workability standards and information on sampling, frequency, and type of testing for conformity control, see KS EAS 131-1 and KS 594.

Workability/consistence of the fresh concrete mix is an important function of both the mix design and the construction. It is one of the key factors in producing a concrete layer with good construction ability and performance. Frequent testing of the concrete delivered to site is essential for quality control.

The workability of the fresh concrete mix can depend upon many factors including the water/cement ratio (a high ratio will give a low strength but very workable mix), aggregate grading and shape, the presence of fibres and/or admixtures, etc.

There is no direct test to measure workability/consistence, however useful indicator tests include:

1. The slump test (BS EN 12350-2). It is reported that the slump test is only suitable for concrete of medium or high workability (i.e., a slump of 25-125 mm). For stiff mixes with low slump values, the slump does not show the difference between concretes with different workability.
2. The Vebe test (BS EN 12350-3). Note that if the Vebe time is less than 5s or more than 30s, then the Vebe test is unsuitable for the concrete consistence.
3. The compacting factor test (aka degree of compactability) (BS EN 12350-4).

The required workability of the fresh concrete mix will need to be different for each type of concrete pavement and the method of laying the concrete. For example:

1. Roller compacted concrete requires a very stiff mix with a slump of practically 0 mm.
2. A mix suitable for slip-form construction will need a stiffer mix (because side forms are not used) requiring a typical slump of 20-40 mm.
3. A mix suitable for mechanised fixed-form paving will need a typical slump of 35-50 mm.
4. A mix suitable for manual fixed-form paving will require a typical slump of 55-65 mm.

8.8.11 Concrete Mix Design

KS EAS 131-1 contains lots of useful information about Concrete Mix Design

It should be understood that there is no unique concrete mix 'recipe' that can be used for all types of concrete pavement. The mix design process is often based on experience with past mixes and knowledge of local aggregates and cements.

When cement is added to the other concrete components, there needs to be enough of each component to produce a concrete layer with good performance. The following factors are key:

1. For fresh concrete – workability, compaction, curing, and no bleeding.
2. For hardened concrete – strength (flexural/compressive), durability, minimal shrinkage, and a skid resistant surface.

The concrete mix design can be complex, as it needs to include many components, including:

1. The aggregate type/size/grading/shape.
2. The cement type/content.
3. Any admixtures.
4. Water content.
5. Any air entrainment.
6. Whether mix in plant method uses volume or mass batching.
7. Method of laying e.g., slip form paver, fixed form paver or fixed form manual, etc.

There are five main stages of concrete mix design as described below and illustrated in Figure 8.10 and Figure 8.11.

Five Main Stages of Concrete Mix Design (UK BRE Design Method).

Stage 1: Deals with strength and leads to selection of the target water /cement ratio.

The standard deviation of the mix can be determined using values that will depend upon the number of samples previously tested and the characteristic strength of the mix.

Firstly, the margin is required to calculate the target mean strength. This can either be specified, or calculated using Calculation C1:

$$\textbf{\textit{Margin}} = \boldsymbol{k}\left(\begin{array}{c}\text{(a value representing the \%}\\ \text{defective permitted below}\\ \text{the characteristic strength)}\end{array}\right)\boldsymbol{x\ s}\ \text{(the std deviation).}$$

Equation C1

The target mean strength (to 2 decimal places) can then be calculated using Calculation C2:

$$\textbf{\textit{Target mean strength = Specified characteristic strength + M (Margin)}}$$

Equation C2

Using Table 2 for the mix strength of a mix made with a free-water/cement ratio of 0.5, the strength class of cement and the aggregate to be used.

This strength value can be plotted in Figure 4 and a new parallel curve drawn until it crosses a horizontal line representing the mean target strength. This will give a free-water/cement ratio and this value can be compared with any maximum value already given. The lowest of these values should be used.

Stage 2: Deals with workability to determine the free-water content (from Table 3), which will depend upon type and max size of aggregate to give the specified slump/(or vebe time).

Stage 3: Combines Stage 1 and 2 results to determine the cement content. This uses calculation C3:

$$\textbf{\textit{Cement content}} = \frac{\textbf{\textit{free-water content}}}{\textbf{\textit{free-water/cement ratio}}}$$

Equation C3

Stage 4: Determine the total aggregate content. Use an estimate of the density of the compacted mix (from Figure 5) depending upon the free-water content and the relative density of the combined aggregate. The total aggregate content can be estimated using Calculation C4:

$$\textbf{\textit{Total aggregate content = Wet density of concrete – Cement content – Free water content.}}$$

Equation C4

Stage 5: Determine the fine and coarse aggregate contents.

This can be done using Figure 6 which shows recommended values for fine aggregate content depending on the maximum size of aggregate, workability level .and the grading of the fine aggregate (passing 600-micron sieve) and the free-water/cement ratio.

The coarse aggregate content can then be calculated using Calculation C5:

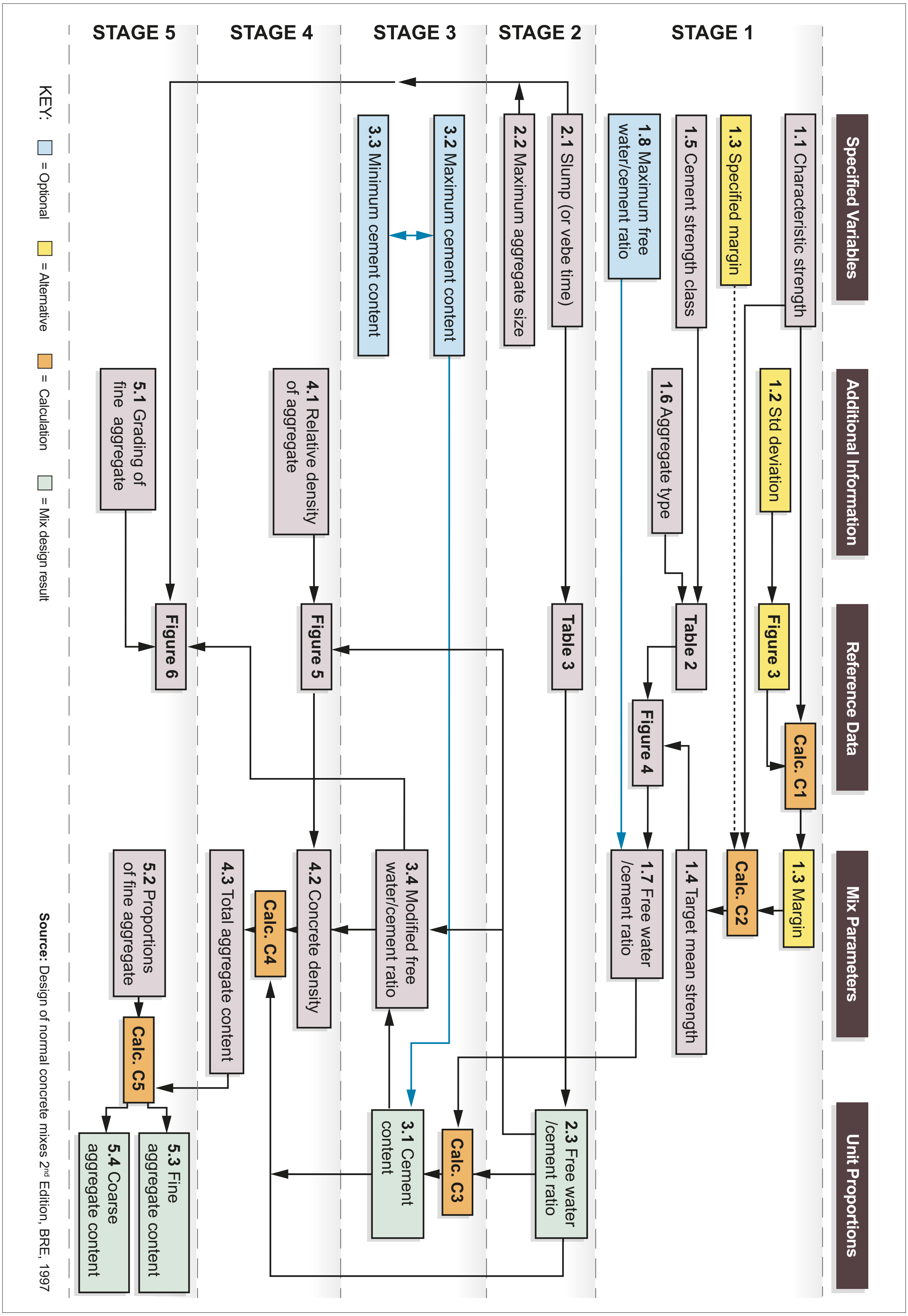
$$\textbf{\textit{Coarse aggregate content = Total aggregate content}}\ \text{(derived in Stage 4)} – \textbf{\textit{Fine aggregate content}}$$

Equation C5

It should be noted that the mix design process should give a satisfactory concrete mix for use in the first trial mix, which should then be adjusted, as required.

After the concrete mix has been designed, it is essential that the contractor carries out laboratory, and

Figure 8.10 Flow Chart Showing 5 Stages of Concrete Mix Design



Source: Design of normal concrete mixes 2nd Edition, BRE, 1997

Figure 8.11 Continuation of the Design Flow Chart Showing 5 Stages of Concrete Mix Design

Table 2 Approximate compressive strengths (N/mm²) of concrete mixes made with a free-water/cement ratio of 0.5

Cement strength class	Type of coarse aggregate	Compressive strengths (N/mm²) Age (days) 3	7	28	91
42.5	Uncrushed	22	30	42	49
	Crushed	27	36	49	56
52.5	Uncrushed	29	37	48	54
	Crushed	34	43	55	61

Throughout this publication concrete strength is expressed in the units N/mm².
1 N/mm² = 1 MN/m² = 1 MPa. (N = newton; Pa = pascal.)

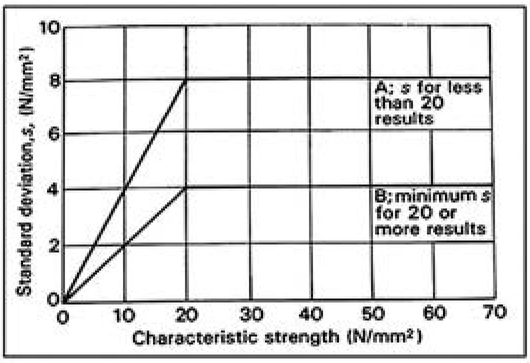


Figure 3: Relationship between standard deviation and characteristic strength.

Table 3 Approximate free-water contents (kg/m³) required to give various levels of workability

Slump (mm)		0–10	10–30	30–60	60–180
Vebe time (s)		>12	6–12	3–6	0–3
Maximum size of aggregate (mm)	Type of aggregate				
10	Uncrushed	150	180	205	225
	Crushed	180	205	230	250
20	Uncrushed	135	160	180	195
	Crushed	170	190	210	225
40	Uncrushed	115	140	160	175
	Crushed	155	175	190	205

Note: When coarse and fine aggregates of different types are used, the free-water content is estimated by the expression:

$\frac{2}{3} W_f + \frac{1}{3} W_c$

where W_f = free-water content appropriate to type of fine aggregate
and W_c = free-water content appropriate to type of coarse aggregate.

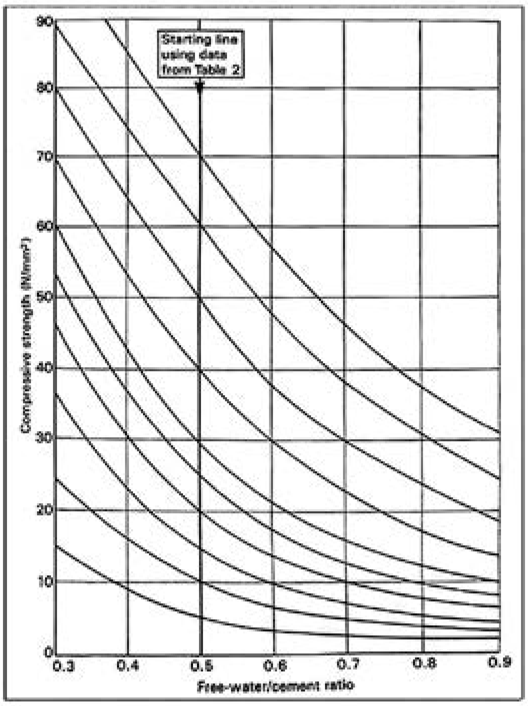


Figure 4: Relationship between compressive strength and free-water/cement ratio.

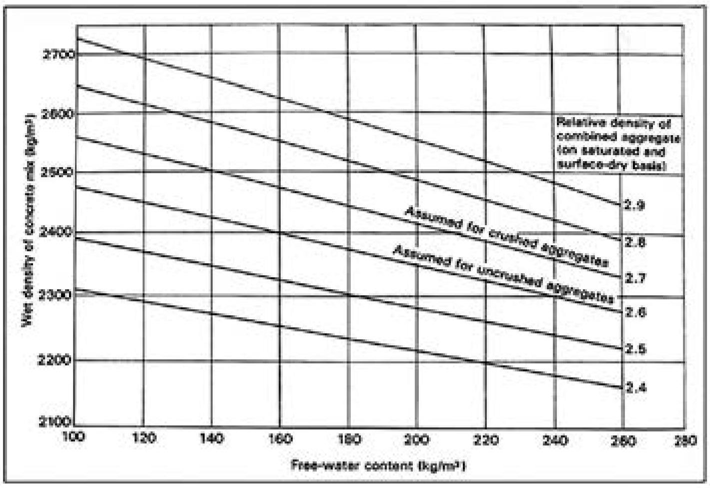


Figure 5: Estimated wet density of fully compacted concrete.

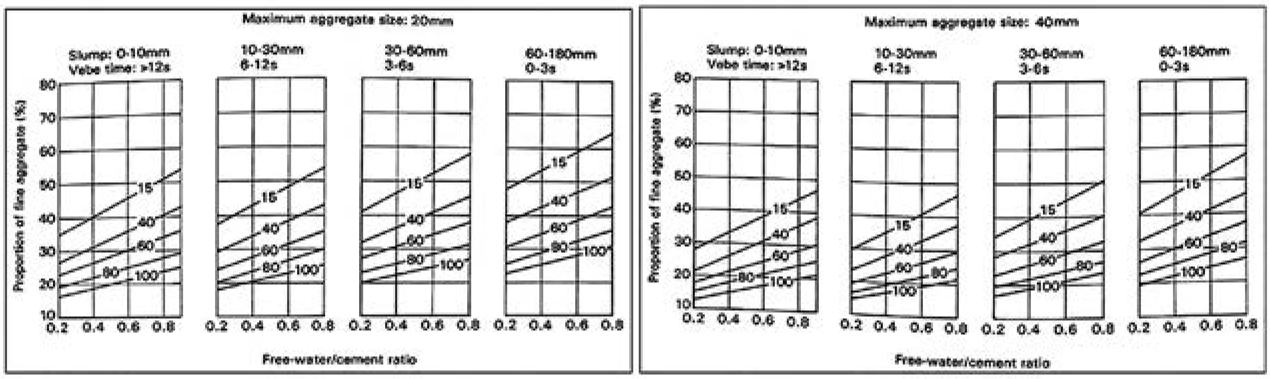


Figure 6: Recommended proportions of fine aggregate according to % passing 600 micron sieve.

then field, trials to evaluate both the fresh, and hardened, properties of the proposed concrete base (and the cement-bound sub-base). The trials will need to demonstrate workability, compactability, slip formability (if required), strength, a skid resistant surface with adequate texture depth, etc.

There isn't enough space in this chapter to cover concrete mix design in detail, but many excellent guides are available, some of which are shown below (alphabetical order by country):

Kenya

- KS EAS 131 Concrete Part 1: Specification, performance, production and conformity.

Australia

- Austroads (2021). Guide to Pavement Technology. Part 4C: Materials for Concrete Road Pavements. AGPT04C-17, Edition 2.1, May 2021. Sydney, Australia: Austroads.

India

- Indian Roads Congress (2017). IRC 44-2017: Guidelines for Concrete Mix Design for Pavements, 3rd Revision, 2017.

South Africa

- South African Pavement Engineering Manual (SAPEM) (2014). Chapter 09: Materials Utilisation and Design. 2nd Edition, 2014.

UK

- MCHW Volume 1: Specification for Highway Works, Series 1000: Road Pavements - Concrete Materials. (For JUC, JRC, CRCP and CRCB the concrete strength class required is usually C40/50, with a minimum of C32/40). This references the following British and European Standards:
  - BS 8500-1: Concrete. Method of specifying and guidance for the specifier.
  - BS 8500-2: Concrete. Specification for constituent materials and concrete.
  - BS EN 206: Concrete. Specification, performance, production and conformity.
  - BS EN 12350: Testing Fresh Concrete.
  - BS EN 12390: Testing Hardened Concrete.
  - BS EN 13877: Concrete Pavements – Materials & Requirements.
  - BRE (Teychenne D.C, Franklin, R. E. et al) (1997). Design of Normal Concrete Mixes. 2nd Edition. Building Research Establishment Ltd. Watford, UK.

USA

- Portland Cement Association (2021). Design & Control of Concrete Mixtures. 17th Edn.
- American Concrete Institute (ACI) (2022). ACI PRC-211.1-22: Selecting Proportions for Normal, and High-Density Concrete guide.
- American Concrete Institute (ACI) (2017). ACI PRC-325.14-17: Guide for Design and Proportioning of Concrete Mixtures for Pavements, 2017 (to supplement PRC-211.1-22).
- AASHTO (2020). PP84 Standard Practice for Developing Performance Engineered Concrete Pavement Mixtures.

During construction, the concrete mix will need frequent sampling and testing to ensure good quality control and a consistency.

8.8.12 Roller Compacted Concrete Design

Cement

The cement specifications for roller compacted concrete (RCC), with equivalent Kenyan Standards, are given in Table 8.6 below as a guide only. There may well be other types of cement available in Kenya (including more environmentally friendly cements) that would be suitable.

Table 8.6 RCC Cement Specifications

Cement and Combinations	Minimum Cement or Combination Content
a. Portland cement CEM1 conforming to KS EAS 18-1.	270 kg/m³
b. CRCB/CRCP Design Portland-slag cement CEMII/A-S and CEMII/B-S conforming to KS EAS 18-1.	270 kg/m³
c. Portland-fly ash cement CEMIIA-V and CEMIIB-V conforming to KS EAS 18-1. • Where CEMIIA-V has a maximum siliceous fly ash content of 20 % and CEMIIB-V has a maximum siliceous fly ash content of 35 %.	
d. Pozzolanic cement CEMIV/A(V) conforming to KS EAS 18-1. • Where CEMIV/A(V) has a maximum siliceous fly ash content of 35 %.	
e. BS 8500-2 combination of Portland cement CEMI with not more than 35 % fly ash for use as a cementitious component in structural concrete conforming to BS EN 450-1.	270 kg/m³

Notes: *KS EAS 18-1: Cement - Part 1: Composition, specification & conformity criteria for common cements*

Aggregate

Crushed gravel and recycled aggregates are not permitted in RCC as aggregate interlock is such an important feature.

If RCC is to have a concrete surface then a maximum coarse aggregate size of 14 mm should be specified to reduce the risk of segregation. If it is to be overlaid by asphalt, then a maximum size of 20 mm may be specified.

RCC has a higher proportion of fine aggregate than conventional concrete allowing better packing and a closed finish to be achieved.

Applied finishes such as brushed or power-floated are generally not possible with RCC due to the low amount of cement paste available.

BS 9227 (Hydraulically bound materials for civil engineering purposes. Specification for production and installation in pavements) contains good advice on the selection of RCC for particular applications and will help to generate confidence for pavement designers and specifiers in the use of RCC.

C40/50 RCC produced in accordance with the Specification could be relied upon to have a flexural strength of 5 MPa at the specified lower level of field density. This permitted a design strength of 5 MPa to be adopted in the development of the RCC pavement design curves. It is important to understand that the RCC tested was strictly in accordance with aggregate type and grading.

RCC aggregate grading is shown in Table 8.7.

1 2 3 4 5 6 7 8 Pavement Materials

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Pavement Materials

Table 8.7 RCC Aggregate Grading

Sieve (mm)	Percent Passing by Mass *			
	0/14		0/20	
	Min	Max	Min	Max
31.5	100	100	100	100
20	100	100	90	100
14	86	100	78	94
10	72	95	62	86
8	68	90	56	80
4	52	74	38	59
2	41	61	28	48
1	30	50	19	39
0.5	20	37	15	31
0.25	11	25	9	23
0.125	6	15	6	15
0.063	2	10	2	10

***Note:* *** *The maximum coarse aggregate size of 20 mm is usually specified for RCC that is to be overlaid with asphalt. A maximum 14 mm can be specified if it is to have a concrete surface (to reduce the risk of segregation).*

Mix Design Approach

There are four recognised methods of mixture proportioning for RCC:

1. Soil Compaction Method.
2. Solid Suspension Model.
3. Optimal Paste Volume Method.
4. Meet Specified Consistency Limits.

These approaches are described in detail in Production of Roller Compacted Concrete (PCA 2006), but the most commonly applied approach for pavements is the Soil Compaction Method, where the mixture is determined by optimum moisture content and maximum dry density. BS 9227 (Hydraulically bound materials for civil engineering purposes. Specification for production and installation in pavements), Annex E outlines this approach. The optimum moisture content and maximum dry density of the mixture should be determined using the vibrating hammer method, detailed in BS EN 13286-4 (Unbound and hydraulically bound mixtures. Test methods for laboratory reference density and water content. Vibrating hammer).

8.9 Materials Classification

8.9.1 General

The materials are classified as follows:

Natural or blended (Mechanically Stabilised) granular materials coded with the letter 'G', followed by a number denoting the minimum CBR strength measured after 4 days' soak on the sample moulded and compacted at OMC to the densities specified in the individual materials charts.

Hydraulically Improved Granular materials (Cement/HRB/Lime) coded with letters 'HIG', followed by a number denoting the minimum CBR strength measured after 7 days cure and 7 days soak.

Hydraulically Modified Stone (Cement/HRB) coded with letters "HMS", followed by a number denoting the minimum UCS in kPa (cylinder strength) measured after 7 days cure and 7 days soak. HRB denotes Hydraulic Road Binder complying with BS EN 13282-1.

Hydraulically Bound Stone (Cement/HRB) coded with letters 'HBS', followed by a number denoting the minimum UCS in kPa (cylinder strength) measured after 7 days cure and 7 days soak.

Bitumen Stabilised Materials coded with letters BSM, followed by a number denoting the minimum indirect tensile strength in kPa tested after 7 days of curing and 7 days of soaking.

The rest of the materials bear the acronym of the material name.

The materials for capping and pavement structures have been classified as indicated in Table 8.8 and detailed in the Material Specification Charts that follow this table.

Table 8.8 Material Codes and Abbreviated Specifications

Code	Material	Application	Chart
Granular Materials (Clayey and Silty Sands, Natural Gravels and Natural Materials blended with Crushed Stone Aggregates)			
G3	Clayey and silty sands of minimum 4 days soak CBR of 3 %.	Embankment fill material only.	GM1
G8	Clayey and silty sands, natural gravels or natural materials blended with up to 20 % stone aggregates of minimum 4 days soak CBR of 8 %.	Lower capping on subgrade class S1 to convert to S2.	GM2
G10	Clayey and silty sands, natural gravels or natural materials blended with up to 30 % stone aggregates of minimum 4 days soak CBR of 10 %.	Capping on subgrade classes S1 and S2 subgrade for foundation Class F1.	GM3
G14	Clayey and silty sands, natural gravels or natural materials blended with up to 30 % stone aggregates of minimum 4 days soak CBR of 14 %.	Capping on subgrade classes S1 to S3 for foundation classes F1 and F2.	GM4
G20	Clayey and silty sands, natural gravels or natural materials blended with up to 30 % stone aggregates of minimum 4 days soak CBR of 20 %.	Gravel Wearing Course material.	GM5
G23	Clayey and silty sands, natural gravels or natural materials blended with up to 30 % stone aggregates of minimum 4 days soak CBR of 23 %.	Capping on subgrade classes S3 and S4 for foundation classes F2 and F3.	GM6
G25	Clayey and silty sands, natural gravels or natural materials blended with up to 30 % stone aggregates of minimum 4 days soak CBR of 25 %.	Sub-base for traffic classes TC0.5 and TC0.25.	GM7
G30	Clayey and silty sands, natural gravels or natural materials blended with up to 30 % stone aggregates of minimum 4 days soak CBR of 30 %.	• Base for traffic classes TC0.1 and TC0.025. • Sub-base for traffic classes TC1, TC3 and TC10.	GM8
G45	Clayey and silty sands, natural gravels or natural materials blended with up to 30 % stone aggregates of minimum 4 days soak CBR of 45 %; OR crushed stone gravel, crusher run of indeterminate CBR but complying with the gradation.	• Capping on subgrade classes S3 to S5 for foundation classes F2 to F4. • Base for Traffic classes TC0.5 and TC0.25, and sub-base for TC1-TC10.	GM9
G80	Natural gravels or natural materials blended with up to 30 % stone aggregates of minimum 4 days soak CBR of 80 %; OR crushed stone gravel, crusher run of indeterminate CBR but complying with the gradation.	Base for Traffic classes TC1, TC3 and TC10, and sub-base for TC17.	GM10
Graded Crushed Stone			
GCS-F	Crushed stone aggregates Class F of minimum 4 days soak CBR 30 %. Mostly weathered rock.	Base for traffic classes TC0.025 and TC0.1.	GM11
GCS-E	Crushed stone aggregates Class E of minimum 4 days soak CBR of 50 %, maximum ACV 35 % and LAA 50 %. Mostly soft stone, and partially weathered rock.	Base for traffic classes TC0.25 and TC0.5, and sub-base for TC1 to TC10	
GCS-D	Crushed stone aggregates Class D of minimum 4 days soak CBR of 80 %, maximum ACV 35 % and LAA 50 %. Mostly partially weathered granites, basalts, and other rocks.	Base for traffic classes TC1, TC3 and TC10, and sub-base for TC17.	
GCS-C	GCS Class C of maximum ACV 30 % and LAA 40 %. Mostly fresh corals, and metamorphic rocks.	Base and Sub-base for traffic classes TC1 and TC3.	
GCS-B	GCS Class B of maximum ACV 28 % and LAA 35 %. Mostly fresh granites, basalts, and other igneous rocks.	• Base and Sub-base for traffic class TC10. • Sub-base for traffic class TC17.	
GCS-A	GCS Class A of maximum ACV 25 % and LAA 30 %. Mostly fresh granites, basalts, and other igneous rocks.	• Base for traffic class TC17. • Sub-base for traffic class TC30 and TC50. • Capping on subgrade classes S5 to S6 for foundation class F5 for traffic class TC80 and above.	

Table 8.8 Material Codes and Abbreviated Specifications *(continued)*

Code	Material	Application	Chart
Graded Crushed Stone *(continued)*			
HPS	Hand Packed Stone of maximum ACV 40 % and LAA 60 %. Mostly fresh trachytes, soft stone, basalts, and other igneous rocks.	Sub-base and Base for traffic classes TC0.25 to TC10.	GM12
MAC	Dry-bound and wet-bound macadam. Complies with parent rock requirements for GCS grades	Base for TC3 to TC30 traffic. Sub-base for TC3 to TC150 and higher	GM13
Hydraulically Improved Materials and Hydraulically Bound Materials			
HIG50	Lime and hydraulically improved granular material of minimum CBR of 50% after 7 days cure and 7 days soak.	Sub-base for traffic class TC0.25 and TC0.5.	HM1
HIG60	Lime and hydraulically improved granular material of minimum CBR of 60% after 7 days cure and 7 days soak.	• Sub-base for traffic classes TC1, TC3 and TC10. • Base for traffic class TC 0.025 and TC0.1.	HM2
HIG100	Lime and hydraulically improved granular materials of minimum CBR of 100% after 7 days cure and 7 days soak.	Base for traffic classes TC0.25 and TC0.5.	HM3
HIG160	Lime and hydraulically improved granular materials of minimum CBR of 160% and UCS 1.0 to 2.0 MPa after 7 days cure and 7 days soak	• Base for traffic class TC1, TC3 and TC10. • Sub-base for traffic class TC17, TC30 and TC50. • Capping on subgrade classes S5 to S6 for foundation class F5 for traffic class TC80 and above.	HM4
HMS1	Hydraulically modified stone of minimum UCS 1.0 MPa and maximum UCS 2.5 MPa after 7-day cure and 7-day soak	• Base for traffic Class TC1, TC3 and TC10. • Sub-base for traffic class TC17, TC30 and TC50. • Capping on subgrade classes S5 to S6 for foundation class F5 for traffic class TC80 and above.	HM5
HBS3	Hydraulically bound stone of minimum UCS 3.0 MPa after 7-day cure and 7-day soak	• Base for traffic class TC17, TC30 and TC50. • Sub-base for traffic class TC80 and higher.	HB1
HBS6	Hydraulically bound stone of minimum UCS 6.0 MPa after 7-day cure and 7-day soak	• Base for traffic class TC17, TC30 and TC50. • Sub-base for concrete pavement for traffic class TC150 and higher.	HB2
HBS9	Hydraulically bound stone of minimum UCS 9.0 MPa after 7-day cure and 7-day soak	• Base for traffic class TC50 and higher. • Sub-base for concrete pavement for traffic class TC150 and higher.	HB3

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Pavement Materials

Table 8.8 Material Codes and Abbreviated Specifications *(continued)*

Code	Material	Application	Chart
Bitumen Stabilised Materials			
BSM50	Bitumen Stabilised Material of minimum soaked ITS of 50 kPa.	Base for TC0.25, TC0.5, and TC1 traffic.	BM1
BSM100	Bitumen Stabilised Material of minimum soaked ITS of 100 kPa.	• Base for traffic class TC3, and TC10. • Sub-base for TC17 and TC30.	BM2
BM175	Bitumen Stabilised Material of minimum soaked ITS of 175 kPa.	• Base for traffic class TC17 and TC30. • Sub-base for TC50 and higher.	BM3
Bituminous Mixes for Road Base and Surfacing			
DBM	Dense Bitumen Macadam of minimum Marshall Stability 9 kN and Modulus 5000 MPa.	Base for traffic class TC17 and higher.	BB1
EME	EME Asphalt of minimum modulus 8000 MPa.	Base for traffic class TC50 and higher.	BB2
SBMa	Sand bitumen mix (silty clayey sand) of minimum Marshall Stability 3.75 kN.	Surfacing for TC1 to TC10.	BB3
SBMb	Sand bitumen mix (clean sand) of minimum Marshall Stability 2.5 kN.	Surfacing for TC1 to TC10.	BB4
DSD	Surface dressing made with single sized aggregates	Surfacing for traffic up to 6000 vpd/lane.	SU1
ESS	Emulsion slurry made with aggregate, cement and emulsion. Should usually be applied as a second or third seal. For LVSRs, it should be used in two layers if it is the only seal.	Surfacing for traffic up to 2000 vpd/lane.	SU2
CMA	Cold mix asphalt made with emulsion and graded stone 0/10 or 0/14	Surfacing for TC0.25, TC0.5, TC1, and TC3.	SU3
OTA	Otta seal made with graded aggregate and soft penetration bitumen or cut-back or emulsion	Surfacing for TC0.25, TC0.5, TC1, and TC3.	SU4
DSS	Sand seal made with clean well graded sand and soft penetration bitumen or cut-back or emulsion. If not a second seal, then it must be applied in at least two layers). Split application encouraged.	Surfacing for traffic up to 500 vpd/lane.	SU5
SAN	Sand bitumen mix of minimum Marshall Stability 3 kN and Modulus 1000 MPa.	Surfacing for TC1 and TC3.	SU6
GAP	Gap graded asphalt of minimum Marshall Stability 3 kN and Modulus 1500 MPa.	Surfacing for TC1 and TC3.	SU7
ACII	Flexible asphalt of minimum Marshall Stability 6 kN and Modulus 2500 MPa.	Surfacing for TC1 and TC3.	SU8
ACI	High stability asphalt concrete of minimum Marshall Stability 9 kN and Modulus 4000 MPa.	Surfacing for TC10 or higher.	SU9
ACIb	High stability asphalt concrete (for binder course) of minimum Marshall Stability 9 kN and Modulus 4000 MPa.	Surfacing for TC10 or higher.	SU10
SMA	Stone mastic asphalt of minimum Marshall Stability 9 kN and Modulus 5000 MPa.	Surfacing for TC50 or higher.	SU11
Cobblestones, Concrete Paving Blocks, and Cement Concrete for Rigid Pavements			
ICB	Interlocking cobblestone paving of minimum UCS 25 MPa	Surfacing for TC0.25, TC0.5, TC1.	PB1
IPB	Interlocking concrete paving blocks of minimum UCS 25 MPa	Surfacing for all levels of traffic.	PB2
CP-1	Concrete for TC1 and lower	Surfacing/Base for TC1 and lower.	CP-1
CP-2	Concrete for jointed unreinforced concrete (JUC), jointed reinforced concrete (JRC), continuously reinforced concrete pavement (CRCP)	Surfacing/Base for TC1 and higher.	CP-2
CP-3	Concrete for continuously reinforced concrete base (CRCB)	Base for TC1 and higher.	CP-3
CP-4	Concrete for roller compacted concrete (RCC)	Base for TC1 and higher.	CP-4

8.9.2 Charts for Natural Gravels

Materials Specification Charts	
GRANULAR MATERIALS	
Chart GM1	G3 Material

Materials Requirements	
Natural silty/sandy/gravelly clays, silty/clayey sands conforming to the following requirement:	
CBR at 100% MDD (AASHTO T99) and 4 days soak (%)	Min 3
Swell (%)	Max. 3.0
Max. size	1/2-layer thickness or 50 mm whichever is lesser
Liquid Limit	Max. 70
Plasticity Index (%)	Max. 50

Application
Fill material only.
Construction Procedures
Minimum thickness of compacted layer: 100 mm, Tolerance ±15 mm.
Cross-fall tolerance: +0.25 %.
LAYING: by grader
COMPACTION: • Using vibratory steel drum rollers • Minimum dry density: 100 % MDD (AASHTO T99) • Compaction moisture content: 75 to 100 % OMC (AASHTO T99) • Maximum thickness compacted in one layer: 250 mm • Thickness tolerance: final compacted thickness shall not be less than the specified by more than 15 mm.

Materials Specification Charts

GRANULAR MATERIALS

Chart GM2	G8 Material

Materials Requirements

Natural silty/sandy/gravelly clays, silty/clayey sands, sands conforming to the following requirement:

CBR at 95% MDD (AASHTO T180) and 4 days soak (%)	Min 8
Swell (%)	Max. 3.0
Max. size	1/2-layer thickness or 50 mm whichever is lesser
Liquid Limit	Max. 70
Plasticity Index (%)	Max. 50

Application

Lower capping on S1 to convert it to S2.

Construction Procedures

Minimum thickness of compacted layer: 100 mm, Tolerance ±15 mm.

Cross-fall tolerance: +0.25 %.

LAYING: by grader

COMPACTION:

- Using vibratory steel drum rollers
- Minimum dry density: 100 % MDD (AASHTO T180)
- Compaction moisture content: 75 to 100 % OMC (AASHTO T180)
- Maximum thickness compacted in one layer: 250 mm
- Thickness tolerance: final compacted thickness shall not be less than the specified by more than 15 mm.

Materials Specification Charts	
GRANULAR MATERIALS	
Chart GM3	G10 Material

Materials Requirements	
Natural or blended silty/sandy/gravelly clays, silty/clayey sands, sands, or gravels conforming to the following requirement:	
CBR at 95% MDD (AASHTO T180) and 4 days soak (%)	Min 10
Swell (%)	Max. 2.5
Max. size	2/3-layer thickness or 80 mm whichever is lesser
Plasticity Index (%)	Max. 30
Liquid Limit	Max. 50
Coefficient of Uniformity	Min 6

Application

Lower capping on S1 and S2 subgrade for foundation class F1.

Construction Procedures

Minimum thickness of compacted layer: 100 mm, Tolerance ±15 mm.

Cross-fall tolerance: +0.25 %.

LAYING: by grader

COMPACTION:

- Using vibratory steel drum rollers
- Minimum dry density: 95 % MDD (AASHTO T180)
- Compaction moisture content: 75 to 100 % OMC (AASHTO T180)
- Maximum thickness compacted in one layer: 150 mm
- Thickness tolerance: final compacted thickness shall not be less than the specified by more than 15 mm.

Materials Specification Charts	
GRANULAR MATERIALS	
Chart GM4	G14 Material

Materials Requirements	
Natural or blended silty and clayey sands, sands, gravels, volcanic gravel (scoria), conforming to the following requirement:	
Gravels	
Max. size (mm)	10 - 50
Passing 0.075 mm sieve (%)	Max. 50
Sands, silty and clayey sands	
Max. size (mm)	0.5 – 10
Passing 0.075 mm sieve (%)	Max 50
All materials	
CBR at 95% MDD (AASHTO T180) and 4 days soak (%)	Min 14
Swell (%)	Max. 1.0
Plasticity Index (%)	Max. 30
Plasticity Modulus	Max. 2500
Organic Matter (%)	Max 2
Coefficient of Uniformity	Min 6

Application
Capping on subgrade classes S1 to S3 for foundation classes F1 and F2.
Construction Procedures
Minimum thickness of compacted layer: 100 mm, Tolerance ±15 mm.
Cross-fall tolerance: +0.25 %.
LAYING: by grader
COMPACTION: • Using vibratory steel drum rollers • Minimum dry density: 95 % MDD (AASHTO T180) • Compaction moisture content: 75 to 100 % OMC (AASHTO T180) • Maximum thickness compacted in one layer: 200 mm • Thickness tolerance: final compacted thickness shall not be less than the specified by more than 15 mm

Materials Specification Charts

GRANULAR MATERIALS

Chart GM5	G20 Material

Materials Requirements

Natural gravel or mixtures of natural gravels, volcanic gravel (scoria) and sand or up to 30% of natural stone, crushed stone, scarified pavement material or milled bituminous pavement material conforming to the following requirement:

Bearing strength & plasticity properties			Grading After Compaction for Gravel Wearing Course		
				% by weight passing	
CBR at 95% MDD (AASHTO T180) and 4 days soak (%)	Min 20		**Sieve** (mm)	**Class 1**	**Class 2**
Swell (%)	Max. 1.0		37.5	-	100
Plasticity Index (%)	**Wet areas**	5 – 20	28	100	95 – 100
	Dry areas	5 – 25	20	95 – 100	85 – 100
Plasticity Modulus	Max. 30		14	80 – 100	65 – 100
			10	65 – 100	55 – 100
			5	45 – 85	35 – 92
			2	30 – 68	23 – 77
			1	25 – 56	18 – 62
			0.425	18 – 44	14 – 50
			0.075	12 – 32	10 – 40

Application

Also used as Gravel Wearing Course (GWC) material.

Construction Procedures

Minimum thickness of compacted layer: 100 mm, Tolerance ±15 mm.

Cross-fall tolerance: +0.25 %.

LAYING: by grader

COMPACTION:
- Using vibratory steel drum rollers
- Minimum dry density: 95 % MDD (AASHTO T180)
- Compaction moisture content: between 80 and 100 % OMC (AASHTO T180)
- Maximum thickness compacted in one layer: 200 mm
- Thickness tolerance: final compacted thickness shall not be less than the specified by more than 15 mm.

Materials Specification Charts	
GRANULAR MATERIALS	
Chart GM6	G23 Material

Materials Requirements	
Natural or blended silty and clayey sands, sands, or gravels conforming to the following requirement:	
Gravels	
Max. size (mm)	10 – 50
Passing 0.075 mm sieve (%)	Max 40
Sands, silty and clayey sands	
Max. size (mm)	0.5 – 10
Passing 0.075 mm sieve (%)	Max 50
All materials	
CBR at 95% MDD (AASHTO T180) and 4 days soak (%)	Min 23
Swell (%)	Max. 1.0
Plasticity Index (%)	Max. 25
Plasticity Modulus	Max. 2500
Organic Matter (%)	Max 2
Coefficient of Uniformity	Min 6

Application

Capping on classes S2 and S3 subgrades for foundation classes F2 & F3.

Construction Procedures

Minimum thickness of compacted layer: 100 mm, Tolerance ±15 mm.

Cross-fall tolerance: +0.25 %.

LAYING: by grader

COMPACTION:

- Using vibratory steel drum rollers
- Minimum dry density: 95 % MDD (AASHTO T180)
- Compaction moisture content: 75 to 100 % OMC (AASHTO T180)
- Maximum thickness compacted in one layer: 200 mm
- Thickness tolerance: final compacted thickness shall not be less than the specified by more than 15 mm.

Materials Specification Charts	
GRANULAR MATERIALS	
Chart GM7	G25 Material

Materials Requirements		
Natural or blended silty and clayey sands, sands, or gravels conforming to the following requirement:		
Natural Gravels		
CBR at 95% MDD (AASHTO T180) and 4 days soak (%)	Min 25	
Swell (%)	Max. 1.0	
Max. size	2/3 of layer thickness or 80 mm whichever is lesser	
Uniformity Coefficient	Min. 6	
Plasticity Index (%)	**Wet areas**	Max 15
	Dry areas	Max 20
Plasticity modulus	Max 250	
Clayey and Silty Sands		
CBR at 95% MDD (AASHTO T180) and 4 days soak (%)	Min 25	
Swell (%)	Max. 1.0	
Passing 2 mm sieve (%)	Max. 95	
Passing 0.075 mm sieve (%)	Min 10 – Max 30	
Uniformity Coefficient	Min. 6	
Plasticity Index (%)	**Wet areas**	Min 5 – Max 15
	Dry areas	Min 5 – Max 20
Plasticity modulus	Max 250	

Application

Sub-base material for TC0.5 and TC0.25 traffic.

Construction Procedures

Minimum thickness of compacted layer: 100 mm, Tolerance ±15 mm.

Cross-fall tolerance: +0.50 %.

LAYING: by grader

COMPACTION:

- Using vibratory steel drum rollers
- Minimum dry density: 95 % MDD (AASHTO T180)
- Compaction moisture content: between 80 and 100 % OMC (AASHTO T180)
- Maximum thickness compacted in one layer: 200 mm
- Thickness tolerance: final compacted thickness shall not be less than the specified by more than 15 mm.

Materials Specification Charts

GRANULAR MATERIALS

Chart GM8	G30 Material

Materials Requirements			
Bearing strength & plasticity properties		**Grading After Compaction**	
CBR at 95% MDD (AASHTO T180) and 4 days soak (%)	Min 30	**Sieve** (mm)	**% by weight passing**
Plasticity Index (%)	Max 12	50	100
Plasticity Modulus	Max 250	37.5	80 – 100
Swell(%)	Max 0.5	28	
		20	60 – 100
		10	
Mechanical Stabilisation		5	30 – 100
These requirements and limitations also apply to mixtures of natural gravel and sand and of natural gravel and up to 30% of stone (crushed or not).		2	20 – 95
		1.18	17 – 75
		0.300	9 – 50
		0.425	7 – 33
		0.075	5 – 25

Application
Sub-base material for TC1, TC3, and TC10, and base for TC0.025 & TC0.1.

Construction Procedures

Minimum thickness of compacted layer: 125 mm, Tolerance ±10 mm.

Cross-fall tolerance: +0.25 %.

LAYING: by grader or better by paver

COMPACTION:
- Using vibratory steel drum rollers
- Minimum acceptable dry density: 95 % MDD (AASHTO T180)
- Compaction moisture content: between 80 and 100 % OMC (AASHTO T180)
- Maximum thickness compacted in one layer: 200 mm
- Thickness tolerance: final compacted thickness shall not be less than the specified by more than 10 mm.

Higher relative compaction may be specified to increase bearing strength, provided the nodule hardness is adequate.

Materials Specification Charts

GRANULAR MATERIALS

Chart GM9	G45 Material

Materials Requirements			
Bearing strength & plasticity properties		**Grading After Compaction**	
CBR at 95% MDD (AASHTO T180) and 4 days soak (%)	Min 45	**Sieve** (mm)	**% by weight passing**
Los Angeles Abrasion (%)	Max. 70	50	100
Aggregate Crushing Value (%)	Max. 40	37.5	90 – 100
Plasticity Index (%)	Max 10	28	75 – 95
Plasticity Modulus	Max 250	20	60 – 90
Mechanical Stabilisation		10	40 – 75
These requirements and limitations also apply to mixtures of natural gravel and sand and of natural gravel and up to 30% of stone (crushed or not)		5	29 – 65
		2	20 – 45
		1.18	17 – 40
		0.425	12 – 31
		0.075	5 – 25

Application

Capping on S3 subgrade for foundation classes F2 to F4. Sub-base for TC1-TC10 and base for TC0.5 & TC0.25 traffic.

Construction Procedures

Minimum thickness of compacted layer: 125 mm, Tolerance ±10 mm.

Cross-fall tolerance: +0.25 %.

LAYING: by grader or better by paver

COMPACTION:

- Using vibratory steel drum rollers
- Minimum acceptable dry density: 95 % MDD (AASHTO T180)
- Compaction moisture content: between 80 and 100 % OMC (AASHTO T180)
- Maximum thickness compacted in one layer: 200 mm
- Thickness tolerance: final compacted thickness shall not be less than the specified by more than 10 mm.

Higher relative compaction may be specified to increase bearing strength, provided the nodule hardness is adequate.

Materials Specification Charts

GRANULAR MATERIALS

Chart GM10	G80 Material

Materials Requirements

Bearing strength & plasticity properties		Grading After Compaction	
CBR at 95% MDD (AASHTO T180) and 4 days soak (%)	Min 80	**Sieve** (mm)	**% by weight passing**
Los Angeles Abrasion (%)	Max. 50	50	100
Aggregate Crushing Value (%)	Max. 35	37.5	95 – 100
Plasticity Index (%)	Max 10	28	80 – 100
Plasticity Modulus	Max 250	20	60 – 100
Mechanical Stabilisation		10	35 – 100
These requirements and limitations also apply to mixtures of natural gravel and sand, and up to 30 % of stone (crushed or not).		5	20 – 95
		2	12 – 80
		1	10 – 65
		0.425	7 – 50
		0.075	4 – 20

Application

Base for TC1, TC3, and TC10, and sub-base for TC17. In areas with annual rainfall less than 500 mm, if permission has been sought and granted by the Chief Engineer Materials, material of CBR 60 % may be used as an alternative to G80 base for TC1.

Construction Procedures

Minimum thickness of compacted layer: 125 mm, Tolerance ±10 mm.

Cross-fall tolerance: +0.25 %.

LAYING: by grader or better by paver

COMPACTION:
- Using vibratory steel drum rollers
- Minimum acceptable dry density: 95 % MDD (AASHTO T180)
- Compaction moisture content: between 80 and 105 % OMC (AASHTO T180)
- Maximum thickness compacted in one layer: 200 mm
- Thickness tolerance: final compacted thickness shall not be less than the specified by more than 10 mm.

Higher relative compaction may be specified to increase bearing strength, provided the nodule hardness is adequate.

8.9.3 Charts for Graded Crushed Stone, Hand Packed Stone, and Macadam

Materials Specification Charts	
GRANULAR MATERIALS AND SOILS	
Chart GM11	**Graded Crushed Stone (GCS)**

Materials Requirements							
GCS Class		**Class A**	**Class B**	**Class C**		**Class D**	**Class E**
Nominal Size (mm)	–	**0/30**	**0/30**	**0/40**	**0/60**	**0/50**	**0/50**
	BS Sieve (mm)	**Percentage by Weight Passing**					
Grading Envelope	75	–	–	–	100	–	–
	63	–	–	–	95 - 100	–	–
	50	–	–	100	85 - 100	100	100
	37.5	100	100	90 -100	75 - 95	90 -100	90 -100
	28	90 -100	90 -100	75 - 95	60 - 87	80 - 100	75 - 95
	20	65 - 95	65 - 95	60 - 90	50 - 80	60 - 100	60 - 90
	10	40 - 70	40 - 70	40 - 75	30 - 67	35 - 90	40 - 75
	6.3	30 - 55	30 - 55	30 - 63	23 - 58	–	–
	5	–	–	–	–	20 - 75	29 - 65
	2	20 - 40	20 - 40	20 - 45	13 - 40	12 - 50	20 - 45
	1	15 - 32	15 - 32	15 - 35	7 - 32	10 - 40	17 - 40
	0.425	10 - 24	10 - 24	10 - 26	4 - 20	7 - 33	12 - 31
	0.075	4 - 10	4 - 10	4 - 12	0 - 10	4 - 20	5 - 12
	Parameter	**Percentage**					
Stone Physical Requirements	LAA Max.	30	35	40	40	50	50
	ACV Max.	25	28	30	30	35	35
	SSS Max.	12	12	12	12	20	–
	FI Max.	20	25	30	30	–	–
	CR Min.	100	100	80	80	–	–
	PI Max.	NP	NP	NP	NP	6	10
	Sand E Min.	40	40	–	–	–	–
	CBR Min.	28	–	–	–	80	45
Elastic Modulus (MPa)	Min	500	500	400	400	350	300
Application	Base	TC17	TC10	TC3&TC1	TC3&TC1	TC1 - TC10	TC0.25 & TC0.5
	Sub-base	TC30&TC50	TC17	TC3&TC1	TC3&TC1	TC17	TC1 - TC10

Construction Procedures							
		Class A	**Class B**	**Class C**		**Class D**	**Class E**
Minimum compacted thickness	125 mm	125 mm	125 mm	125 mm	150mm	125 mm	125 mm
Maximum compacted thickness	200 mm	200 mm	200 mm	200 mm	200 mm	200 mm	200 mm
LAYING	**Paver**					**Paver or Grader**	
COMPACTION		**Percent of MDD (VH)**				**% MDD AASHTO T180**	
Dry Density	Average	98	98	98	96		
	Minimum	96	96	96	94	95	95
		Percent of SG					
	Average	85	85	82	82		
	Minimum	82	82	80	80		
		Percent of OMC (VH)				**% OMC AASHTO T180**	
		80 – 100	80 – 100	80 – 100	80 – 100	80 – 100	17 - 40
GCS Class	Equivalent to G30 material and the specifications are in accordance with Chart GM8.						

Materials Specification Charts

HAND PACKED STONE

Chart GM12	Hand Packed Stone (HPS)

Materials Requirements				
Parent Rock				
Properties		**Class A**	**Class B**	**Class C**
UCS at 7 days soak		Min 20 MPa	Min 10 MPa	Min 3 MPa
CBR at 95% MDD (AASHTO T180) and 4 days soak (%)		Min 80	Min 60	Min 50
Los Angeles Abrasion (%)		Max 50	Max 50	Max 60
Aggregate Crushing Value (%)		Max 35	Max 35	Max 40
Plasticity Index (%)		Max 10	Max 10	Max 10
Application				
Base		TC10	TC3	TC1
Sub-base			TC10	TC1 - TC3
Filler (0/10mm Aggregates)			**Stone Dimensions**	
Sieve Size (mm)	**% Passing**	**Dimension**	**150mm Stone**	**200mm Stone**
10	100	**Length** (mm)	150	200
6.3	90-100	**Width** (mm)	100 - 150	100 - 200
4	75-95	**Breadth** (mm)	100 - 150	100 - 200
2	50-70			
1	33-50			
0.425	20-33			
0.300	16-28			
0.150	10-20			
0.075	6-12			

Traffic Limitations

Sub-base and Base for TC0.25 to TC10 traffic.

Construction Procedures

COMPACTION:

- A Sub-base of minimum G30 must be provided; constructed as other sub-bases.
- Pack stones tightly, wedge stones with smaller stone chips using hammers and steel rods.
- Provide appropriate surfacing of base.

Materials Specification Charts	
DRY AND WET-BOUND MACADAM	
Chart GM13	**Macadam (MAC)**

Materials Requirements			
Parent Rock			
Properties	**Class A**	**Class B**	**Class C**
LAA (%) **Max.**	30	35	40
ACV (%) **Max.**	25	28	30
SSS (%) **Max.**	12	12	12
FI (%) **Max.**	20	25	30
CR Min.	100	100	80
PI (%) **Max.**	NP	NP	NP
Sand E (%) **Min.**	40	40	40
Application			
Base	TC30	TC10	TC3
Sub-base	TC50 – TC150	TC30	TC10
Particle Size Distribution			
Aggregates		**Filler** (0-5mm Aggregates)	
Sieve Size (mm)	**% Passing**	**Sieve Size** (mm)	**% Passing**
75	100	**2**	100
50	85-100	**1**	85-99
37.5	35-70	**0.425**	55-90
28	0-15	**0.150**	25-45
20	0-10	**0.075**	0-10

Construction Procedures

COMPACTION:

- A Sub-base of minimum G30 must be provided. Compact the aggregate in layers not exceeding twice the maximum stone size. Spread filler and vibrate in until no further movement. Brush of any loose fines and continue until design thickness is achieved.
- Minimum roller size should not be less than 10 tonnes.

1 2 3 4 5 6 7 8 Pavement Materials

8.9.4 Charts for Hydraulically Improved Granular Materials and Hydraulically Bound Stone

Materials Specification Charts	
HYDRAULICALLY IMPROVED GRANULAR MATERIAL (HIG)	
Chart HM1	**HIG of Minimum CBR 50 % (HIG50)**

Materials Requirements				
Material Before Treatment			**Cement/HRB**	
Experience has shown that materials which comply with the following requirements are generally suitable for improvement.			Portland cement (CEM I – 42.5 MPa) complying to KS EAS 18-1 or HRB complying with BS EN 13282.	
	Material Classification	Min G14	**Amounts Required**	
Grading for Natural Gravels	**Max. Size**	10 – 15 mm	**Plastic Gravels** (%)	2 – 4
	Passing 0.075mm sieve	Max. 40	**Sands and Clayey Sands** (%)	2 – 3
Grading for Sands, Silty and Clayey Sands	**Max. Size**	0.5 – 10 mm	**Lime**	
	Passing 0.075mm sieve	Max. 50	Hydrated calcium lime *(See standard specification)*	
	Plasticity Index (%)	Max. 30		
	Plasticity Modulus	Max 2,500	**Amounts Usually Required** (%)	1 – 4
	Organic Matter	Max. 2		
Extra Requirements for Lime Treatment	**Passing 0.425mm Sieve**	Min. 15		
	Plasticity Index (%)	Min. 10		
Treated Material				
HIG50: CBR of Laboratory mix at 95% MDD (modified AASHTO) and 7 days cure + 7 days soak: 50 %, PI min 5 % - max 12 %. PM max 250.				

Application
Sub-base for TC0.25 and TC0.5
Construction Procedures
Minimum thickness of compacted layer: 100 mm, Tolerance ±15 mm.
Cross-fall tolerance: ±0.50 %.
MIXING: in place (pulvimixer or travel plant)
LAYING: by grader

COMPACTION:
- Using vibratory steel drum rollers
- Minimum dry density: 95 % MDD (AASHTO T180)
- Compaction moisture content: between OMC-2 and OMC (AASHTO T180)
- Maximum thickness compacted in one layer: 200 mm
- Time allowed to complete compaction and finishing: 2 hrs (cement), 4 hrs (lime)
- Thickness tolerance: final compacted thickness shall not be less than the specified by more than 15 mm.

PROTECTION AND CURING:
- Time allowed to place protection: 8 hours (lime), 4 hours (cement)
- No traffic permitted for first 7 days

Materials Specification Charts

HYDRAULICALLY IMPROVED GRANULAR MATERIAL (HIG)

Chart HM2	HIG of Minimum CBR 60 % (HIG60)

Materials Requirements

Material Before Treatment			Cement/HRB	
Experience has shown that materials which comply with the following requirements are generally suitable for improvement.			Portland cement (CEM I – 42.5 MPa) complying to KS EAS 18-1 or HRB complying with BS EN 13282.	
	Material Classification	G20	**Amounts Required**	
Grading for Natural Gravels	Max. Size	10 – 15 mm	Plastic Gravels (%)	2 – 4
	Passing 0.075mm sieve	Max. 40	Sands and Clayey Sands (%)	2 – 4
Grading for Sands, Silty and Clayey Sands	Max. Size	0.5 – 10 mm	**Lime**	
	Passing 0.075mm sieve	Max. 50	Hydrated calcium lime *(See standard specification)*	
	Plasticity Index (%)	Max. 25		
	Plasticity Modulus	Max 2,500	Amounts Usually Required (%)	2 – 4
	Organic Matter	Max 1,200		
Extra Requirements for Lime Treatment	Passing 0.425mm Sieve	Min. 15		
	Plasticity Index (%)	Min. 10		
Treated Material				
HIG60: CBR of Laboratory mix at 95 % MDD (modified AASHTO) and 7 days cure + 7 days soak: 60 %, PI min 5 % - max 12 %. PM max. 250.				

Application

Sub-base for TC1, TC3, TC10 and base for TC0.025 and TC0.1

Construction Procedures

Minimum thickness of compacted layer: 100 mm, Tolerance ±15 mm.

Cross-fall tolerance: ±0.50 %

MIXING: in place (pulvimixer or travel plant)

LAYING: by grader

COMPACTION:
- Using vibratory steel drum rollers
- Minimum dry density: 95 % MDD (AASHTO T180)
- Compaction moisture content: between OMC-2 and OMC (AASHTO T180)
- Maximum thickness compacted in one layer: 200 mm
- Time allowed to complete compaction and finishing: 2hrs (cement), 4hrs (lime)
- Thickness tolerance: final compacted thickness shall not be less than the specified by more than 15 mm

PROTECTION AND CURING:
- Time allowed to place protection: 8 hours (lime), 4 hours (cement)
- No traffic permitted for first 7 days

Materials Specification Charts	
HYDRAULICALLY IMPROVED GRANULAR MATERIAL (HIG)	
Chart HM3	HIG of Minimum CBR 100 % (HIG100)

Materials Requirements

Material Before Treatment			Cement/HRB	
Experience has shown that materials which comply with the following requirements are generally suitable for improvement.			Portland cement (CEM I – 42.5 MPa) complying to KS EAS 18-1 or HRB complying with BS EN 13282.	
	Material Classification	G25	Amounts Required	
Grading for Natural Gravels	Max. Size	10 – 50 mm	Plastic Gravels (%)	2 – 4
	Passing 0.075mm sieve	5 – 35	Sands and Clayey Sands (%)	2 – 3
Grading for Sands, Silty and Clayey Sands	Max. Size	1 – 10 mm	Lime	
	Passing 0.075mm sieve	Max. 40	Hydrated calcium lime *(See standard specification)*	
	Plasticity Index (%)	Max. 20		
	Plasticity Modulus	Max 250	Amounts Usually Required (%)	2 – 4
	Organic Matter	Max. 1		
Extra Requirements for Lime Treatment	Passing 0.425mm Sieve	Min. 15		
	Plasticity Index (%)	Min. 10		
Treated Material				
HIG100: CBR of Laboratory mix at 95 % MDD (modified AASHTO) and 7 days cure + 7 days soak: 100 %, PI max 8 %. PM max. 250. Other properties as indicated in Table 5.1				

Application

Base for TC0.25 and TC0.5 traffic

Construction Procedures

Minimum thickness of compacted layer: 125 mm, Tolerance ±10 mm

Cross-fall tolerance: +0.25 %

MIXING: in place by grader or by pulvimixer or travel plant. Then apply water by a bowser capable of spraying water evenly over the surface.

LAYING: by grader

COMPACTION:
- Using vibratory steel drum rollers
- Minimum dry density: 95 % MDD (AASHTO T180)
- Compaction moisture content: between OMC-2 and OMC (AASHTO T180)
- Maximum thickness compacted in one layer: 200 mm
- Time allowed to complete compaction and finishing: 2 hrs (cement), 4 hrs (lime)
- Thickness tolerance: final compacted thickness shall not be less than the specified by more than 10 mm

PROTECTION AND CURING:
- Time allowed to place protection: 8 hours (lime), 4 hours (cement)
- No traffic permitted for first 7 days

Materials Specification Charts

HYDRAULICALLY IMPROVED GRANULAR MATERIAL (HIG)	
Chart HM4	**HIG of Minimum CBR 160 % (HIG 160)**

Materials Requirements

Material Before Treatment

Experience has shown that materials which comply with the following requirements are generally suitable for improvement.

	Material Classification	G30
Grading for Natural Gravels	**Max. Size**	10 – 50 mm
	Passing 0.075 mm sieve	5 – 35
Grading for Sands, Silty and Clayey Sands	**Max. Size**	1 – 10 mm
	Passing 0.075 mm sieve	Max. 40
	Plasticity Index (%)	Max. 12
	Plasticity Modulus	Max 250
	Organic Matter	Max. 1
Extra Requirements for Lime Treatment	**Passing 0.425 mm Sieve**	Min. 15
	Plasticity Index (%)	Min. 10

Cement/HRB

Portland cement (CEM I – 42.5 MPa) complying to KS EAS 18-1 or HRB complying with BS EN 13282.

Amounts Required	
Plastic Gravels (%)	2 – 4
Sands and Clayey Sands (%)	2 – 4

Lime

Hydrated calcium lime
(See standard specification)

Amounts Usually Required (%)	2 – 4

Treated Material

HIG160: CBR of Laboratory mix at 95 % MDD (modified AASHTO) and 7 days cure + 7 days soak: 160 %,
PI max 6 %. PM max. 250.

Application

Sub-base for TC17, TC30, and TC50 and base for TC1, TC3, and TC10. Capping on subgrade classes S5 to S6 for foundation class F5 for traffic class TC 80 and above.

Construction Procedures

Minimum thickness of compacted layer: 125 mm, Tolerance ±10 mm

Cross-fall tolerance: +0.25 %

MIXING: in place by grader or by pulvimixer or travel plant. Then apply water by a bowser capable of spraying water evenly over the surface.

LAYING: by grader

COMPACTION:
- Using vibratory steel drum rollers
- Minimum dry density: 95 % MDD (AASHTO T180)
- Compaction moisture content: between OMC-2 and OMC (AASHTO T180)
- Maximum thickness compacted in one layer: 200 mm
- Time allowed to complete compaction and finishing: 2 hrs (cement), 4 hrs (lime)
- Thickness tolerance: final compacted thickness shall not be less than the specified by more than 10 mm.

PROTECTION AND CURING:
- Time allowed to place protection: 8 hours (lime), 4 hours (cement)
- No traffic permitted for first 7 days

Materials Specification Charts	
HYDRAULICALLY MODIFIED STONE (HMS)	
Chart HM5	HMS of Minimum UCS 1.0 MPa (HMS1)

Materials Requirements	
Material Before Treatment	**Cement/HRB**
Experience has shown that materials which comply with the following requirements are generally suitable for improvement.	Portland cement (CEM I – 42.5 MPa) complying to KS EAS 18-1 or HRB complying with BS EN 13282.
Material Classification	**Estimated Amounts Required**
Min G45 or GCS-A to GCS-D	1 – 2 % for GCS-A, GCS-B, GCS-C. 2 – 3 % for GCS-D or G45.
Treated Material	
HIG160: CBR of Laboratory mix at 95 % MDD (modified AASHTO) and 7 days cure + 7 days soak: 160 %, PI max 6 %. PM max. 250.	
UCS (MPa):	7 days cure + 7 days soak 1.0 – 2.5 (max) MPa (Cylinder) or minimum CBR 160 %
Plasticity Index (%) after 7 days soak	<6
Plasticity Modulus after 7 days soak	<250

Application

Sub-base for TC17, TC30, and TC50 and base for TC1, TC3, and TC10. Capping on subgrade classes S5 to S6 for foundation class F5 for traffic class TC80 and above.

Construction Procedures

Minimum thickness of compacted layer: 125 mm, Tolerance ±10 mm

Cross-fall tolerance: +0.25 %

MIXING: Stationary plant or pulvimixer

LAYING: Paver

COMPACTION:
- Using vibratory steel drum rollers
- Minimum dry density: 95 % MDD (AASHTO T180)
- Compaction moisture content: between OMC-2 and OMC (AASHTO T180)
- Maximum thickness compacted in one layer: 200 mm
- Time allowed to complete compaction and finishing: 2 hrs (cement or HRB)
- Thickness tolerance: final compacted thickness shall not be less than the specified by more than 10 mm

PROTECTION AND CURING:
- Time allowed to place protection: 4 hours (cement or HRB)
- No traffic permitted for first 7 days

Materials Specification Charts	
HYDRAULICALLY BOUND STONE (HBS)	
Chart HB1	**HBS of Minimum UCS 3.0 MPa (HBS3)**

Materials Requirements	
Material Before Treatment	**Cement/HRB**
Experience has shown that materials which comply with the following requirements are generally suitable for improvement.	Portland cement (CEM I – 42.5 MPa) complying to KS EAS 18-1 or HRB complying with BS EN 13282.
Material Classification	**Estimated Amounts Required**
Min G45 or GCS-A to GCS-D	2 – 3 % for GCS-A, GCS-B, GCS-C. 3 – 4 % for GCS-D of G45.
Treated Material	
UCS (MPa):	7 days cure + 7 days soak 3 – 6 MPa (Cylinder)
Strength ratio soaked/OMC	>0.8
Plasticity Index (%) **after 7 days soak**	<6
Plasticity Modulus after 7 days soak	<250

Application

Sub-base for TC80 and higher, and base for TC17, TC30 and TC50. Capping on subgrade classes S5 to S6 for foundation class F5 for traffic class TC 80 and above.

Construction Procedures

Minimum thickness of compacted layer: 125 mm, Tolerance ±10 mm.

Cross-fall tolerance: +0.25 %

MIXING: Stationary plant or pulvimixer

LAYING: Paver

COMPACTION:
- Using vibratory steel drum rollers
- Minimum dry density: 95 % MDD (AASHTO T180)
- Compaction moisture content: between OMC-2 and OMC (AASHTO T180)
- Maximum thickness compacted in one layer: 200 mm
- Time allowed to complete compaction and finishing: 2 hrs (cement or HRB),
- Thickness tolerance: final compacted thickness shall not be less than the specified by more than 10 mm

PROTECTION AND CURING:
- Time allowed to place protection: 4 hours (cement or HRB)
- No traffic permitted for first 7 days

Materials Specification Charts	
HYDRAULICALLY BOUND STONE (HBS)	
Chart HB2	HBS of Minimum UCS 6.0 MPa (HBS6)

Materials Requirements	
Material Before Treatment	**Cement/HRB**
Experience has shown that materials which comply with the following requirements are generally suitable for improvement.	Portland cement (CEM I – 42.5 MPa) complying to KS EAS 18-1 or HRB complying with BS EN 13282.
Material Classification	**Estimated Amounts Required**
Min GCS-A, GCS-B and GCS-C	3 – 4 % for GCS-A, GCS-B and GCS-C.
Treated Material	
UCS (MPa):	7 days cure + 7 days soak 6 – 9 MPa (Cylinder)
Strength ratio soaked/OMC	> 0.8
Plasticity Index (%) **after 7 days soak**	< NP
Plasticity Modulus after 7 days soak	< N/A

Application

Sub-base for TC150 and higher, and base for TC17, TC30 and TC50.

Construction Procedures

Minimum thickness of compacted layer: 125 mm, Tolerance ±10 mm.

Cross-fall tolerance: +0.25 %

MIXING: Stationary plant or pulvimixer

LAYING: Paver

COMPACTION:
- Using vibratory steel drum rollers
- Minimum dry density: 95 % MDD (AASHTO T180)
- Compaction moisture content: between OMC-2 and OMC (AASHTO T180)
- Maximum thickness compacted in one layer: 200 mm
- Time allowed to complete compaction and finishing: 2hrs (cement or HRB),
- Thickness tolerance: final compacted thickness shall not be less than the specified by more than 10 mm

PROTECTION AND CURING:
- Time allowed to place protection: 4 hours (cement or HRB)
- No traffic permitted for first 7 days

Materials Specification Charts	
HYDRAULICALLY BOUND STONE (HBS)	
Chart HB3	HBS of Minimum UCS 9.0 MPa (HBS9)

Materials Requirements			
Material Before Treatment		**Cement/HRB**	
Experience has shown that materials which comply with the following requirements are generally suitable for improvement.		Portland cement (CEM I – 42.5 MPa) complying to KS EAS 18-1 or HRB complying with BS EN 13282.	
Material of GCS-A		**Estimated Amounts Required**	4 – 5 %
Stone Grading		**Coarse Aggregate (mm)**	>2
Sieve Size (mm)	**% Passing**	**Crushing Ratio** (%)	80
37.5	100	**Flaxiness Index** (%)	25
28	90 – 100	**Los Angeles Abrasion** (%)	35
20	65 – 95	**Aggregate Crushing Value** (%)	Max 28
10	40 – 70	**Combined Aggregate Requirements**	
6.3	30 – 55	**Fines (Passing 0.425mm)**	Non-plastic
2	18 – 40	**Sand Equivalent** (%)	Min. 40
1	13 – 32	**S.S.S** (%)	12
0.425	9 – 24	**Organic Matter** (%)	0.3
0.075	0 - 6		
Treated Material			
UCS (MPa):	7 days cure + 7 days soak 9 – 15 MPa (Cylinder Strength). No result less than 6 MPa, No strength to exceed 15 MPa.		
Strength ratio soaked/OMC	>0.8		

Application

Sub-base for TC50 and higher. Sub-base for concrete pavement for traffic class TC150 and higher.

Construction Procedures

Minimum thickness of compacted layer: 125 mm, Tolerance ±10 mm.

Cross-fall tolerance: +0.25 %

MIXING: Stationary plant

LAYING: Paver

COMPACTION:
- Using vibratory steel drum rollers
- Dry Density: 96 % target density established in the test BS 5835 and 85 % specific gravity of stone (Oven-dry value)
- Maximum thickness compacted in one layer: 200 mm
- Time allowed to complete compaction and finishing: 2hrs (cement or HRB),
- Thickness tolerance: final compacted thickness shall not be less than the specified by more than 10 mm.

PROTECTION AND CURING:
- Time allowed to place protection: 4 hours (cement or HRB)
- No traffic permitted for first 7 days.
- Protection by bituminous seal coat (preferably emulsion)

8.9.5 Charts for Bitumen Stabilised Materials

Materials Specification Charts	
BITUMEN STABILISED MATERIALS (BSM)	
Chart BM1	**BSM of Minimum ITS 50 kPa (BSM50)**

Materials Requirements

Material Before Treatment	
Material Class	Min G30, or recycled asphalt pavement
CBR after 4-day soak	Min 30 %
Plasticity Index	Max 12 %
Organic Mater	Max 1 %
Grading Modulus	1.2 – 2.7

Bitumen Stabiliser and Active Filler
Bitumen Binder: Slow setting A3 and A4 anionic and K3 cationic emulsions, foamed bitumen.
Lime for Pre-Treatment and Active Filler
Hydrated Calcium Lime: See standard specification **Amounts Usually Required:** pre-treatment 1 to 2 % and maximum of 1 % as active filler.
Cement for Active Filler
Portland cement (CEM I – 42.5 MPa) complying to KS EAS 18-1 or HRB complying with BS EN 13282. **Amounts required:** Max 1 %

Recommended Residual Binder Content	
Material	**Residual Bitumen** (%)
Reclaimed Asphalt Pavement	1.75 – 2.50
Graded Crushed Rock	2.25 – 3.00
Gravels of CBR ≥ 30 %	2.5 – 3.25
Gravels/sands of CBR ≥ 20 %	2.5 – 4.0

Treated Material

The mix shall comply with the following specifications:

Property	Specimen curing regime & moisture conditions	Specification (%)
ITS dry (kPa)	100 mm diameter Marshall specimen cured for 72 hours at 40°C	125 - 225
ITS wet (kPa)	100 mm diameter Marshall specimen cured for 72 hours at 40°C and soaked for 24 hours	50 - 100
UCS soaked (kPa)	UCS specimen cured for 72 hours at 40°C and soaked for 4 days	Min 500
CBR soaked (%)	CBR specimen cured for 72 hours at 40°C and soaked for 4 days	Min 50
TSR	Tensile Strength Ratio:(ITSwet/ ITSdry)× 100 %	Min 50%

Application

Base for TC0.25, TC0.5, and TC1 traffic.

Construction Procedures

Use bitumen emulsion A4-60 %. Dilute with suitable water (40 % of bitumen content of emulsion).
Minimum permissible residual bitumen content is 2.0 %.

Minimum thickness of compacted layer: 100 mm, Tolerance ±10 mm.

Cross-fall tolerance: +0.25 %.

MIXING: In place by travel plant, pulvimixer, disc harrow or grader (least preferred) and stationary batching in a plant or labour based using hand tools i.e. pans.

LAYING: by grader, paver or labour. Laying should not be done in wet weather conditions.

COMPACTION:
- Using vibratory steel drum rollers
- Minimum dry density: 95 % MDD (Vibratory hammer)
- Maximum thickness compacted in one layer: 200 mm
- Thickness tolerance: final compacted thickness shall not be less than the specified by more than 10 mm.

PROTECTION AND CURING:
- Not applicable, allows immediate trafficking

Materials Specification Charts

BITUMEN STABILISED MATERIALS (BSM)

Chart BM2	BSM of Minimum ITS 100 kPa (BSM100)

Materials Requirements

Material Before Treatment		Bitumen Stabiliser and Active Filler	
Material Class	Min G45, or recycled asphalt pavement	**Bitumen Binder:** Slow setting A3 and A4 anionic and K3 cationic emulsions, foamed bitumen.	
		Lime for Pre-Treatment and Active Filler	
CBR after 4-day soak	Min 45 %	**Hydrated Calcium Lime:** See standard specification **Amounts Usually Required:** pre-treatment 1 to 2 % and maximum of 1 % as active filler.	
Plasticity Index	Max 10 %		
Organic Mater	Max 1 %		
Grading Modulus	1.2 – 2.7	**Cement for Active Filler**	
		Portland cement (CEM I – 42.5 MPa) complying to KS EAS 18-1 or HRB complying with BS EN 13282. **Amounts required:** Max 1 %	
		Recommended Residual Binder Content	
		Material	**Residual Bitumen** (%)
		Reclaimed Asphalt Pavement	1.75 – 2.50
		Graded Crushed Rock	2.25 – 3.00
		Gravels of CBR ≥ 30 %	2.5 – 3.25
		Gravels/sands of CBR ≥ 20 %	2.5 – 4.0

Treated Material

The mix shall comply with the following specifications:

Property	**Specimen curing regime & moisture conditions**	**Specification** (%)
ITS dry (kPa)	100 mm diameter Marshall specimen cured for 72 hours at 40°C	200 - 275
ITS wet (kPa)	100 mm diameter Marshall specimen cured for 72 hours at 40°C and soaked for 24 hours	100 - 150
TSR	Tensile Strength Ratio:(ITSwet/ ITSdry) ×100 %	Min 60

Application

Base for TC3 and TC10 traffic. Sub-base for TC17 and TC30.

Construction Procedures

Use bitumen emulsion A4-60 %. Dilute with suitable water (40 % of bitumen content of emulsion) or use foamed bitumen. Minimum permissible residual bitumen content is 2.0 %.

Minimum thickness of compacted layer: 100 mm, Tolerance ±10 mm.

Cross-fall tolerance: +0.25 %.

MIXING: In place by travel plant (if foamed bitumen is used then the machine should be capable of foaming).

LAYING: by grader, paver or labour. Laying should not be done in wet weather conditions.

COMPACTION:
- Using vibratory steel drum rollers
- Minimum dry density: 95 % MDD (Vibratory hammer)
- Maximum thickness compacted in one layer: 200 mm
- Thickness tolerance: final compacted thickness shall not be less than the specified by more than 10 mm.

PROTECTION AND CURING:
- Not applicable, allows immediate trafficking

Materials Specification Charts

BITUMEN STABILISED MATERIALS (BSM)

Chart BM3	BSM of Minimum ITS 175 kPa (BSM175)

Materials Requirements

Material Before Treatment		Bitumen Stabiliser and Active Filler	
Material Class	Min GCS-C, or recycled asphalt pavement	**Bitumen Binder:** Slow setting A3 and A4 anionic and K3 cationic emulsions, foamed bitumen.	
		Lime for Pre-Treatment and Active Filler	
CBR after 4-day soak	Min 80 %	**Hydrated Calcium Lime:** See standard specification **Amounts Usually Required:** pre-treatment 1 to 2 % and maximum of 1 % as active filler.	
Plasticity Index	Max 6 %		
Organic Mater	Max 1 %		
Grading Modulus	1.2 – 2.7	**Cement for Active Filler**	
		Portland cement (CEM I – 42.5 MPa) complying to KS EAS 18-1 or HRB complying with BS EN 13282. **Amounts required:** Max 1 %	
		Recommended Residual Binder Content	
		Material	**Residual Bitumen** (%)
		Reclaimed Asphalt Pavement	1.75 – 2.50
		Graded Crushed Rock	2.25 – 3.00
		Gravels of CBR ≥ 30 %	2.5 – 3.25
		Gravels/sands of CBR ≥ 20 %	2.5 – 4.0

Treated Material

The mix shall comply with the following specifications:

Property	Specimen curing regime & moisture conditions	Specification (%)
ITS dry (kPa)	100 mm diameter Marshall specimen cured for 72 hours at 40 °C	≥275
ITS wet (kPa)	100 mm diameter Marshall specimen cured for 72 hours at 40 °C and soaked for 24 hours	≥175
TSR	Tensile Strength Ratio:(ITSwet/ ITSdry) ×100 %	Min 70

Application

Base for TC17 and TC30 traffic. Sub-base for TC50 and higher.

Construction Procedures

Use bitumen emulsion A4-60 %. Dilute with suitable water (40 % of bitumen content of emulsion) or use foamed bitumen. Minimum permissible residual bitumen content is 2.0 %.

Minimum thickness of compacted layer: 100 mm, Tolerance ±10 mm.

Cross-fall tolerance: +0.25 %.

MIXING: In place by travel plant (if foamed bitumen is used then the machine should be capable of foaming).

LAYING: by grader, paver or labour. Laying should not be done in wet weather conditions.

COMPACTION:
- Using vibratory steel drum rollers
- Minimum dry density: 95 % MDD (Vibratory hammer)
- Maximum thickness compacted in one layer: 200 mm
- Thickness tolerance: final compacted thickness shall not be less than the specified by more than 10 mm.

PROTECTION AND CURING:
- Not applicable, allows immediate trafficking

8.9.6 Charts for Bituminous Mixes for Bases and Surfacing

Materials Specification Charts	
BITUMINOUS BASE MATERIALS	
Chart BB1	**Dense Bitumen Macadam (DBM)**

Materials Requirements				
Bituminous Binder: 30/50, 50/70, and 80/100 penetration grade bitumen at a content rate of minimum 4 % determined through mix design.				
Aggregate Grading			**Coarse Aggregate > 2 mm**	
Sieve Size (mm)	**Nominal Size (mm)**		**Crushing Ratio** (%)	100
	37.5	**25**	**LAA Max.**	30
50	100		**ACV Max.**	25
37.5	90 – 100	100	**SSS Max.** (%)	12
25	– 90	90 – 100	**FI Max.** (%)	20
19		– 90	**Fine Aggregate** (mm) < 2	
12.5			**Sand Equivalent Min.**(%)	40
9.5			**Mineral Filler**	
6.3			Cement, Lime, Limestone or other mineral matter. Shall be non-plastic	
2.36	15 - 41	19 - 45		
1.18			**Passing 0.425 mm** (%)	100
0.300			**Passing 0.075 mm** (%)	75
0.150			**Bulk Density in Toluene**	0.5 -0.9 g/ml
0.075	0 - 6	1 - 7		
Usual Bit. (%)	Min. 4	Min. 4		

Nominal Maximum Aggregate Size (mm)	37.5	25.0	**Notes:** An aggregate gradation that passes below the PCS control point is classified as 'coarse graded'. A gradation that passes above the PCS control point is classified as 'fine graded'.
Primary Control Sieve (mm)	9.5	4.75	
PCS Control Point (% passing)	47	40	

Mix Requirements (See Appendix D and E)

MARSHALL:
- Compactibility Test as in Part 1 of BS 5835 to be used in choosing a bitumen content for a trial mix.
- Voids in total mix: 3 % – 8 %.
- As an alternative to the above grading, the Bailey Method may be used to determine a suitable grading.
- **Trial Pavement Mixes:** From Cores: Voids in compacted mixes: Maximum 7 % (average of at least 5 cores).
  - Marshall Stability: 9 kN at 2 x 75 blows, Flow = 2-4 mm, Dust/Binder ratio = 0.6 to 1.2.
  - Indirect tensile strength: Min. 800 kPa at 25 °C.
  - Tensile strength ratio (wet/dry): Min. 80 %.
- APPLICATION: Base material for TC17 and higher.

SUPERPAVE:
- Compaction levels: Ninitial, Ndesign, and Nmaximum – See Appendix E.
- Compactibility Test as in AASHTO R35 for choosing a trial mix (See Appendix E).
- Voids in total mix: 4 %.
- Grading as per the above envelopes.
- **Trial Pavement Mixes:** From Cores: Voids in compacted mixes: Maximum 6 % (average of at least 5 cores).
  - Density: 96 % @ Ndesign and 98 % @ Nmaximum (See Appendix E).

Application

Surfacing for TC80 and higher.

Performance Requirements for TC80 and Higher

Hamburg Wheel Tracking Test: Max. rut = 15 mm at 60 °C after 10000 passes (5000 cycles).
Fatigue test (4-point bending beam)**:** Min. ratio of final stiffness to initial stiffness after 1 million cycles at 400 microstrain = 50 %.
Dynamic Creep Test: Min. Creep Modulus at 40°C = 10 MPa.

Construction Procedures

MIXING: In stationary plant, bitumen temperatures: 80/100 (130 - 150°C); For Superpave dependent on selected binder.

(...Continues on next page)

Construction Procedures *(...continued)*
LAYING: By mechanical paver at minimum temperatures 80/100 (120 °C - 150 °C); For Superpave dependent on selected binder.
COMPACTION: • Mean density: 95 % of refusal density (no individual result should be below 93%). • Minimum temperature at end of compaction: 80°C (60/70) and 70 °C (80/100); For Superpave dependent on selected binder. • Steel wheel roller (5000-7000 kg/m of roll width). • Pneumatic tyred roller (Min. 4 tonnes per wheel). • Vibratory rollers (Min. 200 kg/m of roll width). • Minimum compacted layer thickness: 60 mm (19 and 12.5 NMAS) and 75 mm (25 and 37.5 NMAS). • Maximum compacted layer thickness: 100 mm (19 and 12.5 NMAS) and 125 mm (19 and 12.5 NMAS).

Materials Specification Charts

BITUMINOUS BASE MATERIALS

Chart BB2	**Enrobé À Module Élevé** (High Modulus Asphalt) **(EME)**

Materials Requirements

Bituminous Binder: 10/20 or 20/30 penetration grade bitumen at a content rate of minimum 4 % determined through mix design.

Aggregate Grading				Coarse Aggregate > 2 mm	
Sieve Size (mm)	**Nominal Size** (mm)			**Crushing Ratio** (%)	100
	0/10	**0/14**	**0/20**	**LAA Max.**	35
50				**ACV Max.**	28
40				**SSS Max.** (%)	12
31.5	1		100	**FI Max.** (%)	25
20		100	90 - 99	**Fine Aggregate** (mm) < 2	
14	100	90 - 99	70 - 95	**Sand Equivalent Min.**(%)	40
10	90 - 99	-	55 - 90	**Mineral Filler**	
6.3	60 - 80	42 - 65	42 - 75	Cement, Lime, Limestone or other mineral matter. Shall be non-plastic	
4	35 - 65	-	-		
2	27 - 42	19 - 42	18 - 35	**Passing 0.425 mm** (%)	100
0.250	8 - 18	8 - 18	8 - 18	**Passing 0.075 mm** (%)	75
0.063	5 - 9	5 - 9	5 - 9	**Bulk Density in Toluene**	0.5 -0.9 g/ml
Usual Bit. (%)	Min. 5.5	Min. 5.3	Min. 5.1		
Gyratory Comp.	80	100	120		

Mix Requirements

Hamburg Wheel Tracking Test: Maximum proportional deformation (%) = 7.5 at 60 °C.
Minimum laboratory dynamic modulus (4-point bending beam): = 8000 MPa @ 20 °C and 5 Hz.
Voids in total mix: 3 % - 5 %.

Application

Base material for TC50 and higher.

Construction Procedures

MIXING: In stationary plant, bitumen temperatures: To be determined for specific sources.

LAYING: By mechanical paver at minimum temperatures: To be determined for specific sources.

COMPACTION:
- Mean density: 96 % maximum theoretical density determined in the laboratory (no individual result should be below 93 %).
- Minimum temperature at end of compaction: To be determined for specific sources.
- Steel wheel roller (5000-7000 kg/m of roll width).
- Pneumatic tyred roller (Min. 4 tonnes per wheel).
- Vibratory rollers (Min. 200 kg/m of roll width).
- Minimum compacted layer thickness: 60 mm.
- Maximum compacted layer thickness: 100 mm.

Materials Specification Charts	
BITUMINOUS BASE MATERIALS	
Chart BB3	**Sand Bitumen Mixture (SBM)** - Bitumen Stabilised Silty Clayey Sand

Materials Requirements			
Silty Clayey Sand		**Bituminous Binder**	
Free of soluble salts, organic and other deleterious matter.		Anionic emulsion A3 or cut-back MC 250 or MC 800	
Passing 0.075 mm (%)	Max. 10 - 30	**Usual Amount** (%)	2.5 - 5 (residual)
Liquid Limit	Max. 40	**SSS Max.** (%)	12
Plasticity Index (%)	Max 15	**FI Max.** (%)	20

Treated Material		
Parameter	**Traffic Class/Specification**	
	TC10	TC3-TC1
Marshall Stability (kN): Minimum	5.25	3.75
Flow Value (mm)	2 - 4	2 - 5
Water Absorbed after 7 days soak (Max %)	4	4

Application

Suitable for TC1 to TC10 (where risk of overloaded axles is low).

Construction Procedures

MIXING: In place, bitumen temperatures (pulvimixer or travel plant).

LAYING: By grader.

COMPACTION:
- Mean density: 95 % Laboratory design Marshall density (no individual result should be below 93%).
- Minimum thickness compacted in one layer: 200 mm.
- Minimum thickness of compacted layer: 100 mm.

Materials Specification Charts	
BITUMINOUS BASE MATERIALS	
Chart BB4	**Sand Bitumen Mixture (SBM)** - Clean Sand

Materials Requirements			
Bituminous Binder: 60/70 or 40/50 penetration grade bitumen at a content 3 % – 4.5 % rate determined through mix design.			
Sand		**Bituminous Binder**	
Free of soluble salts, organic and other deleterious matter.		Bitumen 40/50 or 60/70 pen or emulsion A2 or A3	
Passing 0.075 mm (%)	Max. 20	**Usual Amount** (%)	3 - 4 (residual)
Sand Equivalent (%)	Min. 30	**Mineral Filler**	
		Cement, Lime, Limestone or other mineral matter. Shall be non-plastic	
		Passing 0.425 mm (%)	100
		Passing 0.075 mm (%)	75
Sand Mix Requirements (mm)		TC10	TC3-TC1
Marshall Stability (kN): Minimum		3	2.5
Flow Value (mm)		2 - 4	2 - 5

Application

Suitable for TC1 to TC10 (where risk of overloaded axles is low).

Construction Procedures

MIXING: In stationary plant, bitumen temperatures: 60/70 (130 – 150°C); 80/100 (120 - 140°C).

LAYING: By mechanical paver at minimum temperatures 130°C (60/70) and 125°C (80/100) or by labour-based methods.

COMPACTION:
- Mean density: 95% Laboratory design Marshall density (no individual result should be below 93%).
- Minimum temperature at end of compaction: 80°C (60/70) and 90°C (40/50).
- Thickness of compacted layer: 100-150 mm.

Materials Specification Charts

SURFACING MATERIALS

Chart SU1	Surface Dressing

Materials Requirements

Bituminous Binder: Cationic emulsion K1-70 and 80/100, 150/200 penetration grade bitumen, MC 300, polymer-modified bitumens.

Chippings Grading				
Sieve Size (mm)	Nominal Size (mm)			
	14/20	10/14	6/10	3/6
28	100	–	–	–
20	85 – 100	100	–	–
14	0 – 30	85 – 100	100	–
10	0 – 7	0 – 30	85 – 100	100
6.3	–	0 – 7	0 – 30	85 - 100
5	–	–	0 – 10	–
3	–	–	–	0 – 30
2	0 – 2	0 – 2	0 – 2	0 – 10
0.5	–	–	–	0 – 2

Chippings Requirements				
Chipping Class	1	2	3	4
LAA Max.	20	25	30	35
ACV Max.	16	20	23	26
SSS Max. (%)	12	12	12	12
FI Max. (%)	20	20	25	25
Angularity (all classes)				
Minimum size of stone to be at least 4 times maximum size of chippings.				
Cleanliness (all classes)				
Passing 0.075 mm (%)	1			
Free of deleterious matter.				

Crushed Rock Sand (wholly derived from crushed stone)

Sieve Size (mm)	**% Passing by Weight**
6.30	100
0.300	0 – 15
0.150	0 – 2

Traffic Limitation				
Total traffic on 2 lanes (veh./day) **in year of application of chippings**	>6,000	2,000 – 6,000	500 – 2,000	<500
Chippings class required	1	2	3	4

Construction Procedures

Use of K1-70 bitumen emulsion should only be considered if other options are not possible. It requires very high spray rates to attain the specified residual binder content which makes it flow off the surface during spraying on steep and elevated sections. Economically, it has high ex-factory and transport cost components and therefore only viable if produced on site. 80/100 PG bitumen may be blended with up to 5% kerosene depending on road surface temperature to enable uniform spray rate.

SPRAYING:
- By bitumen distributor.
- Spraying temperatures: K1 – 70: 75 – 85°C; and 80/100 PG : 160 - 170°C.
- The rate of application shall not vary by more than ±10%.

SPREADING THE CHIPPINGS:
- By mechanical spreader.
- Time allowed (after spraying) to spread chippings: 1 minute.
- The rate of application shall not vary by more than ±10%.

ROLLING:
- Preferably by pneumatic tyred rollers (minimum 1 tonne per wheel).
- Steel wheeled rollers of less than 8 tonnes accepted.
- Time allowed (after spraying) to start rolling: 2 minutes.

TRAFFIC CONTROL:
- Speed must be restricted to 30 km/hr until chippings are held.

Materials Specification Charts

SURFACING MATERIALS

Chart SU2	Emulsion Slurry Seal

Materials Requirements

Anionic Emulsion:
A4-60 (slow-setting slurry), or Cationic emulsion K1-60 (quick-setting slurry), K3-65 (slow-setting slurry)

Aggregate Grading

Sieve size (mm)	% by Weight Passing		
	Type I (Fine)	Type II (Normal)	Type III (Coarse)
10	–	–	100
6.3	–	100	80 – 95
5	–	90 – 100	70 – 90
2	100	60 – 87	40 – 65
1	60 – 85	40 – 67	25 – 45
0.425	30 – 48	22 – 38	15 – 28
0.300	25 – 42	18 – 30	12 – 25
0.150	15 – 30	10 – 20	7 – 18
0.075	10 - 20	5 - 15	5 - 15

Aggregate

Aggregate	
Shall be free of organic and other deleterious matter.	
Sand Equivalent	Min. 40
Percentage of Crusher Dust (all classes)	
Slurry Class A (%)	Min. 50
Slurry Class B (%)	Min. 25
Mineral Filler	
Cement complying to KS EAS 18-1, HRB complying to BS EN 13282, Hydrated Lime or other non-plastic mineral matter	
Usual Amount (%)	1 (by weight)

Application

For traffic up to 2000 vpd/lane. Should usually be applied as a second or third seal. For LVSRs, it should be used in two layers if it is the only seal.

Mixture

EMULSION: 15 – 25 % (by weight of dry aggregate)
WATER : 10 – 25 % (by weight of dry aggregate)

Rate of Application

130 – 250 m^2/m^3 (4 – 8 litre/m^2)

Construction Procedures

MIXING: By concrete mixer or preferably slurry machine.

LAYING: By slurry machine.

CURLING: No traffic until cured to a firm condition (no pick-up by tyres).

ROLLING: If required, by pneumatic tyred roller.

NOTE: *The wet track abrasion test gives some guidance for initial emulsion content, and application rate selection. However, for all roads carrying substantial traffic it is considered necessary to observe the performance of trial sections under traffic before selecting a job mix and application rate.*

Materials Specification Charts	
SURFACING MATERIALS	
Chart SU3	Cold Mix Asphalt Seal

Materials Requirements

Cationic Emulsion: K3-65 (slow setting) or anionic emulsion A3, and suitable aggregates.

Aggregate Grading			Aggregate Requirements	
Sieve Size (mm)	% By Weight Passing		**LAA Max.**	40
	0/10	**0/14**	**ACV Max.**	30
20	100	100	**SSS Max.** (%)	12
14	100	90 - 100	**FI Max.** (%)	25
10	90 - 100	70 - 95	**Sand Equivalent**	Min 40
6.3	62 - 90	55 - 85	**Binder Content**	
4	50 - 80	46 - 75	**Sand Equivalent Min.**(%)	40
2	35 - 65	35 - 60	**Properties of Compacted Mix**	
1	25 - 50	25 - 45	Modified Marshall Stability at 50 blows, 24 hr oven cure at 40°C and 1 hr soak (N)	Min 3000
0.425	14 - 33	14 - 32		
0.300	11 - 27	11 - 27		
0.150	6 - 17	6 - 17	**Voids in Total Mix** (%)	3 – 8
0.075	3 - 8	3 - 8	**Flow** (mm)	2 – 5
Mineral Filler			**Stability loss after immersion** (%)	Max 50
Cement, Lime or other non-plastic materials			**Aggregate Coating** (%)	Min 50

Application

Suitable for traffic classes TC0.25, TC0.5 & TC1& TC3

Construction Procedures

MIXING: In a stationary plant, concrete mixer or in pans using labour.

LAYING: By paver and labour using guide rails.

CURLING: No traffic until cured to a firm condition (no pick-up by tyres). Quarry dust or sand may be applied to facilitate immediate passage of traffic.

ROLLING: Preferably a combination of pneumatic tyred roller and steel drum roller maximum 8 tonne.

Materials Specification Charts

SURFACING MATERIALS

Chart SU4	Otta Seal

Materials Requirements

Bituminous Binder: MC 3000 and 150/200 Pen. Grade Bitumen

Aggregate Grading		Aggregate Requirements		
Sieve Size (mm)	**% By Weight Passing**	**Test Designation.**	vpd at construction <100	vpd at construction >100
		10% FACT (Dry)	90 kN	110 kN
20	100	**Wet/Dry Strength Ratio**	0.60	0.75
14	65 – 100	**LAA Max.**	Max 40	
10	45 – 95	**ACV Max.** (%)	Max 30	
6.3	25 – 80	**Plasticity Index Max.** (%)	Max 10	
4	15 – 65	**FI Max.** (%)	Max 30	
2	10 – 50			
1	5 – 40			
0.425	3 – 30			
0.075	0 – 10			

Traffic Limitations			
Total traffic on 2 lanes single carriageway in both directions (veh. / day) in year of application of aggregates. Surfacing for TC0.25, TC0.5 & TC1& TC3	<100	100 – 1000	>1000
Bitumen Binder	MC 3000	150/200 Pen	150/200 Pen

Construction Procedures

SPRAYING:
- By bitumen distributor.
- Spraying temperatures: MC 3000: 135 – 155 °C; 150/200 Pen. Grade: 165 – 180 °C.
- The rate of application shall not vary by more than ±10%.

SPREADING THE CHIPPINGS:
- By mechanical spreader.
- Time allowed (after spraying) to spread chippings: 5 minutes.
- The rate of application shall not vary by more than ±10%.

ROLLING:
- Preferably by pneumatic tyred roller (minimum 1 tonne per wheel) at hottest 2 hrs of the day every day for first 4 days.
- Steel wheeled roller of less than 8 tonnes accepted for maximum of 4 passes only.
- Time allowed (after spraying) to start rolling: 2 minutes.

TRAFFIC CONTROL:
- Speed must be restricted to 30 km/hr until aggregates are held.

Materials Specification Charts	
SURFACING MATERIALS	
Chart SU5	Sand Seal

Materials Requirements			
Bituminous Binder: MC 800, MC 3000 & K1-70.			
Aggregate Grading		**Aggregate Requirements**	
Sieve Size (mm)	**% Passing**	**Sand Equivalent.**	Max. 40
		Plasticity Index (%)	Non-plastic
6.3	100	**Residue Binder Amount**	0.9 to 1.2 litre/m^2
5	95 - 100	**Sand Spread Rate**	6 to 7 litre/m^2
4	90 - 100		
2	50 - 95		
1	20 - 80		
0.6	10 - 50		
0.425	3 - 25		
0.3	0 - 15		
0.15	0 - 8		
0.075	0 - 5		

Traffic Application	
Total traffic on 2 lanes single carriageway in both directions (veh. /day).	
Less than 100 vpd	Sand seal is applied directly on the base layer. (2 seals preferred).
100 to 500 vpd	Second seal on Surface Dressing or Otta Seal. Split application preferred for better performance.

Construction Procedures

SPRAYING:
- By bitumen distributor.
- Spraying temperatures: MC 800: 110 – 135 °C; MC 3,000: 135 – 155 °C; Emulsions at ambient temperature
- The rate of application shall not vary by more than ±10%.

SPREADING THE CHIPPINGS:
- By labour based methods
- Time allowed (after spraying) to spread chippings: 1 minute.
- The rate of application shall not vary by more than ±10%.

ROLLING:
- Preferably by pneumatic tyred roller (minimum 1 tonne per wheel) at hottest 2 hrs of the day every day for first 2 days.
- Steel wheeled roller of less than 8 tonnes accepted for maximum of 4 passes only.
- Time allowed (after spraying) to start rolling: 5 minutes.

TRAFFIC CONTROL:
- Speed must be restricted to 30 km/hr until aggregates are held.

Materials Specification Charts

SURFACING MATERIALS

Chart SU6	Sand Asphalt

Materials Requirements

Bituminous Binder: 60/70 or 80/100 penetration grade bitumen at a content 6 % – 10 % rate determined through mix design.

Aggregate Grading	
Sieve Size (mm)	**% Passing**
10	100
6.3	95 - 100
2	70 - 100
1	47 - 95
0.425	20 - 75
0.300	15 - 60
0.150	8 - 30
0.075	4 - 12
Uniformity Coefficient	Min. 5

Aggregate	
Sand Equivalent	40
SSS Max. (%)	12
Mineral Filler	
Cement, Lime, Limestone or other mineral matter. Shall be non-plastic	
Passing 0.425 mm (%)	100
Passing 0.075 mm (%)	75
Bulk Density in Toluene	0.5 - 0.9 g/ml
Sand Asphalt Mix Requirements	
Marshall Stability (kN)	3 – 9
Flow Value (mm)	2 – 6
Voids in Total Mix (%)	5 – 10

Application

Sand asphalt is suitable for TC1 to TC3 (where risk of overloaded axles is low).

Construction Procedures

MIXING:
- In stationary plant, bitumen temperatures: 60/70 (130 – 150 °C); 80/100 (120 - 140 °C)

LAYING:
- By mechanical paver at minimum temperatures 130 °C (60/70) and 125 °C (80/100) or by labour-based methods.

COMPACTION:
- Minimum density: 96 % Laboratory design Marshall density.
- Minimum temperature at end of compaction: 80 °C (60/70) and 70 °C (80/100).
- Steel wheel roller (5-7 kg/mm of roll width).
- Pneumatic tyred roller (Min. 2 tonnes per wheel).
- Minimum compacted layer thickness: 25 mm.

Materials Specification Charts

SURFACING MATERIALS

Chart SU7	Gap Graded Asphalt

Materials Requirements

Bituminous Binder: 60/70 penetration grade bitumen at a content rate determined through mix design.

Aggregate Grading		Coarse Aggregate > 2 mm			
Sieve Size (mm)	**Nominal Size Coarse Aggregate** (mm)	**Aggregate Class**	**a**	**b**	**c**
		LAA Max. (%)	30	35	40
19	100	**ACV Max.** (%)	25	28	30
12.5	38 - 100	**SSS Max.** (%)	12	12	12
9.5	0 - 69	**FI Max.** (%)	20	25	25
2.36	0 - 2	**Fine Aggregate < 2 mm**			
Sieve Size (mm)	**Fine Aggregate**	**Sand Equivalent** (%)	40		
		SSS Max. (%)	12		
1.18	100	**Mineral Filler**			
0.425	70 - 97	Cement, Lime, Limestone or other mineral matter. Shall be non-plastic			
0.300	49 - 93	**Passing 0.425 mm** (%)	100		
0.150	16 - 58	**Passing 0.075 mm** (%)	75		
0.075	0 - 20	**Bulk Density in Toluene**	0.5 -0.9 g/ml		

Gap Mix Requirements

Marshall Stability (kN)	3 – 9
Flow Value (mm)	2 – 6
Marshall Quotient (kN/mm)	2.0
Voids in mortar of mix (%)	3 – 9
Filler/bitumen ratio	0.9 – 1.3

MIX DESIGN:

1. Prepare mixture of fine aggregate and filler to give 1:6 ratio by mass of filler to fine aggregate retained on 0.075 mm sieve. Determine the optimum bitumen content which gives maximum Marshall Quotient.
2. By adding coarse aggregate and reducing bitumen content, the Marshall Quotient may be increased to the specified value. 20 % coarse aggregate will increase quotient by a factor of approx. 1.5, 30 % by 1.9, 40 % by 2.4 and 55 % by 3.0. For an increase in coarse aggregate the revised bitumen content is given by:
3. Nominal bitumen content = (Optimum bitumen content (100-S)/100) + 2.3(S)/100. Where S = % of coarse aggregate added.
4. Prepare mixes in accordance with BS 594 at nominal bitumen content from (2) to check specified requirements.

Application

Surfacing for TC1 and TC3.

Construction Procedures

MIXING:

- In stationary plant, bitumen temperatures: 60/70 (130 – 150 °C); 80/100 (120 - 140 °C)

LAYING:

- By mechanical paver at minimum temperatures 130 °C (60/70) and 125 °C (80/100).

COMPACTION:

- Mean density: 96 % Laboratory design Marshall density (no individual result should be below 93%).
- Minimum temperature at end of compaction: 80 °C (60/70) and 70 °C (80/100).
- Steel wheel roller (5-7 kg/mm of roll width).
- Pneumatic tyred roller (Min. 2 tonnes per wheel).
- Minimum compacted layer thickness: 25 mm.

Materials Specification Charts

SURFACING MATERIALS

Chart SU8	Asphalt Concrete (ACII) Continuously Graded

Materials Requirements

Bituminous Binder: 60/70 or 80/100 penetration grade bitumen at a content rate of 5.5% – 7.5% determined through mix design.

Aggregate Grading			Coarse Aggregate > 2 mm			
Sieve Size (mm)	**Nominal Size** (mm) **Coarse Aggregate**		**Aggregate Class**	a	b	c
	0/10	**0/14**	**LAA Max.** (%)	30	35	40
28	-	-	**ACV Max.** (%)	25	28	30
20	-	100	**SSS Max.** (%)	12	12	12
14	100	90 - 100	**FI Max.** (%)	20	25	25
10	90 - 100	70 - 95	**Fine Aggregate < 2 mm**			
6.3	62 - 90	55 - 85	**Sand Equivalent** (%)	40		
2	35 - 65	35 - 60	**SSS Max.** (%)	12		
1	25 - 50	25 - 45	**Mineral Filler**			
0.425	14 - 33	14 - 32	Cement, Lime, Limestone or other mineral matter. Shall be non-plastic.			
0.300	11 - 27	11 - 27	**Passing 0.425 mm** (%)	100		
0.150	6 - 17	6 - 17	**Passing 0.075 mm** (%)	75		
0.075	3 - 8	3 - 8	**Bulk Density in Toluene**	0.5 -0.9 g/ml		

ACII Mix Requirements	
Crushing Ratio (%)	60
Marshall Stability (kN)	4 – 7
Flow Value (mm)	2 – 5
Voids in total mix (%)	3 – 8

Application

Surfacing for TC1 and TC3.

Construction Procedures

MIXING:
- In stationary plant, bitumen temperatures: 60/70 (130 – 150 °C); 80/100 (120 - 140 °C)

LAYING:
- By mechanical paver at minimum temperatures 130 °C (60/70) and 125 °C (80/100).

COMPACTION:
- Mean density: 96 % Laboratory design Marshall density (no individual result should be below 93%).
- Minimum temperature at end of compaction: 80 °C (60/70) and 70 °C (80/100).
- Steel wheel roller (5-7 kg/mm of roll width).
- Pneumatic tyred roller (Min. 2 tonnes per wheel).
- Minimum compacted layer thickness: 25 mm.

Materials Specification Charts

SURFACING MATERIALS

Chart SU9	**Asphalt Concrete Surfacing** – Wearing Course (AC-WC)

Materials Requirements

Bituminous Binder: 60/70, and 80/100 penetration grade bitumen at a content rate of 5.5% – 7.0% determined through mix design. For Superpave, the binder is dependent on analysis of temperature data for the study site.

Aggregate Grading			**Coarse Aggregate > 2 mm**	
Sieve Size (mm)	**Nominal Size** (mm)		**Crushing Ratio** (%)	100
	12.5	**9.5**	**LAA Max.** (%)	30
50			**ACV Max.** (%)	25
37.5			**SSS Max.** (%)	12
25			**FI Max.** (%)	20
19	100		**Fine Aggregate < 2 mm**	
12.5	90 - 100	100	**Sand Equivalent** (%)	Min 40
9.5	90	90 - 100	**Mineral Filler**	
6.3			Cement, Lime, Limestone or other mineral matter. Shall be non-plastic.	
2.36	28 - 58	32 - 67		
1			**Passing 0.425 mm** (%)	100
0.300			**Passing 0.075 mm** (%)	75
0.150			**Bulk Density in Toluene**	0.5 -0.9 g/ml
0.075	2 - 10	2 - 10		
Usual Bit. (%)	Min. 5.5%	Min. 5.5%		

Nominal Maximum Aggregate Size (mm)	12.5	9.5	**Notes:** An aggregate gradation that passes below the PCS control point is classified as "coarse graded". A gradation that passes above the PCS control point is classified as "fine graded".
Primary Control Sieve (mm)	2.36	2.36	
PCS Control Point (% passing)	39	47	

Mix Requirements (See Appendix D and E)

- APPLICATION: Surfacing for TC10 and higher.

SUPERPAVE:

- Compaction levels: Ninitial, Ndesign, and Nmaximum – See Appendix E.
- Compactibility Test as in AASHTO R35 for choosing a trial mix (See Appendix E).
- Voids in total mix: 4 %.
- Grading as per the above envelopes.
- **Trial Pavement Mixes:** From Cores: Voids in compacted mixes: Maximum 6 % (average of at least 5 cores).
  - Density: 96 % at Ndesign and 98 % at Nmaximum (See Appendix E).

Performance Requirements for TC80 and Higher

Hamburg Wheel Tracking Test: Max. rut = 15 mm at 60 °C after 10,000 passes (5,000 cycles).
Fatigue test (4-point bending beam)**:** Min. ratio of final stiffness to initial stiffness after 1 million cycles at 400 microstrain = 50 %.
Dynamic Creep Test: Min. Creep Modulus at 40°C = 10 MPa.

Construction Procedures

MIXING: In stationary plant, bitumen temperatures: 80/100 (130 - 150 °C); For Superpave dependent on selected binder.

(...Continues on next page)

Construction Procedures *(...continued)*
LAYING: By mechanical paver at minimum temperatures 80/100 (120 °C - 150 °C); For Superpave dependent on selected binder.
COMPACTION: • Mean density: 95 % of refusal density; (no individual result should be below 93 %); For Superpave: 94 % theoretical maximum density • Minimum temperature at end of compaction: 80 °C (60/70) and 70 °C (80/100); For Superpave dependent on selected binder • Steel wheel roller (5000-7000 kg/m of roll width) • Pneumatic tyred roller (Min. 4 tonnes per wheel) • Vibratory rollers (Min. 200 kg/m of roll width) • Minimum compacted layer thickness: 60 mm (19 and 12.5 NMAS) and 75 mm (25 and 37.5 NMAS)

Materials Specification Charts

SURFACING MATERIALS

Chart SU10	**Asphalt Concrete Surfacing** – Binder Course (AC-BC)

Materials Requirements

Bituminous Binder: 30/50, 60/70, and 80/100 penetration grade bitumen at a content rate of 5.0% – 6.5% determined through mix design. For Superpave, the binder is dependent on analysis of temperature data for the study site.

Aggregate Grading

Sieve Size (mm)	Nominal Size (mm)			
	25	**19**	**12.5**	**9.5**
50				
37.5	100			
25	90 - 100	100		
19	90	90 - 100	100	
12.5			90 - 100	100
9.5			90	90 - 100
6.3				
2.36	19 - 45	23 - 49	28 - 58	32 - 67
1				
0.300				
0.150				
0.075	1 - 7	2 - 8	2 - 10	2 - 10
Usual Bit. (%)	Min. 5.0 %	Min. 5.0 %	Min. 5.5 %	Min. 5.5 %

Coarse Aggregate > 2 mm	
Crushing Ratio (%)	100
LAA Max. (%)	30
ACV Max. (%)	25
SSS Max. (%)	12
FI Max. (%)	20
Fine Aggregate < 2 mm	
Sand Equivalent (%)	Min 40
Mineral Filler	
Cement, Lime, Limestone or other mineral matter. Shall be non-plastic.	
Passing 0.425 mm (%)	100
Passing 0.075 mm (%)	75
Bulk Density in Toluene	0.5 - 0.9 g/ml

Nominal Maximum Aggregate Size (mm)	25.0	19.0	12.5	9.5
Primary Control Sieve (mm)	4.75	4.75	2.36	2.36
PCS Control Point (% passing)	40	47	39	47

Notes: An aggregate gradation that passes below the PCS control point is classified as 'coarse graded'. A gradation that passes above the PCS control point is classified as 'fine graded'.

Mix Requirements (See Appendix D and E)

APPLICATION: Surfacing for TC10 and higher.
SUPERPAVE:

- Compaction levels: Ninitial, Ndesign, and Nmaximum – See Appendix E.
- Compactibility Test as in AASHTO R35 for choosing a trial mix (See Appendix E).
- Voids in total mix: 4 %.
- Grading as per the above envelopes.
- **Trial Pavement Mixes:** From Cores: Voids in compacted mixes: Maximum 6 % (average of at least 5 cores).
  - Density: 96 % at Ndesign and 98 % at Nmaximum (See Appendix E).

Performance Requirements for TC80 and Higher

Hamburg Wheel Tracking Test: Max. rut = 15 mm at 60 °C after 10,000 passes (5,000 cycles).
Fatigue test (4-point bending beam)**:** Min. ratio of final stiffness to initial stiffness after 1 million cycles at 400 microstrain = 50 %.
Dynamic Creep Test: Min. Creep Modulus at 40 °C = 10 MPa.

Construction Procedures

MIXING: In stationary plant, bitumen temperatures: 80/100 (130 - 150 °C); For Superpave dependent on selected binder.

Construction Procedures *(...continued)*

LAYING: By mechanical paver at minimum temperatures 80/100 (120 °C - 150 °C); For Superpave dependent on selected binder.

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COMPACTION:

- Mean density: 95 % of refusal density (no individual result should be below 93 %); For Superpave: 94 % theoretical maximum density
- Minimum temperature at end of compaction: 80 °C (60/70) and 70 °C (80/100); For Superpave dependent on selected binder
- Steel wheel roller (5000-7000 kg/m of roll width)
- Pneumatic tyred roller (Min. 4 tonnes per wheel)
- Vibratory rollers (Min. 200 kg/m of roll width)
- Minimum compacted layer thickness: 100 mm (19 and 12.5 NMAS) and 125 mm (19 and 12.5 NMAS)

Materials Specification Charts

SURFACING MATERIALS

Chart SU11	Stone Mastic Asphalt (SMA)

Materials Requirements

Bituminous Binder: Polymer modified binder. 40/50 or 60/70 penetration grade bitumen at a content rate of minimum 5% determined through mix design.

Aggregate Grading				Coarse Aggregate > 2 mm	
Sieve Size (mm)	**Nominal Size** (mm)			**Crushing Ratio** (%)	100
	0/9.5	**0/12.5**	**0/19**	**LAA Max.** (%)	35
50				**ACV Max.** (%)	28
37.5				**SSS Max.** (%)	12
25			100	**FI Max.** (%)	25
19		100	90 - 100	**Fine Aggregate < 2 mm**	
12.5	100	90 - 100	50 - 88	**Sand Equivalent** (%)	Min 40
9.5	90 - 95	50 - 80	25 - 60	**Mineral Filler**	
6.7				Cement, Lime, Limestone or other mineral matter. Shall be non-plastic.	
4.75	30 - 50	20 - 35	20 - 28		
2.36	20 - 30	16 - 24	16 - 24	**Passing 0.425 mm** (%)	100
1.18	- 21				
0.300	- 15		-	**Passing 0.075 mm** (%)	75
0.075	8 - 12	8 - 11	8 - 11	**Bulk Density in Toluene**	0.5 - 0.9 g/ml
Usual Bit. (%)	Min. 6.2 %	Min. 5.8 %	Min. 5.4 %		

Mix Requirements

- Designed in similar way as Asphalt Concrete Type II
- Marshall Stability: Min. 6 kN
- Marshall Quotient: > 2.5
- Voids in mineral aggregate: > 17
- Voids in total mix: 3 % - 5 %
- Voids filled with bitumen: 71 % - 92 %
- ITS wet/dry ratio: 70 %
- Dust to binder ratio = 1.0-1.5
- Binder film thickness: >8 μm
- Fibre content: 0.3% to 0.5%

Performance Requirements for TC80 and Higher

Hamburg Wheel Tracking Test: Max. rut = 15 mm at 60°C after 10,000 passes (5,000 cycles).
Fatigue test (4-point bending beam)**:** Min. ratio of final stiffness to initial stiffness after 1 million cycles at 400 microstrain = 50 %.
Dynamic Creep Test: Min. Creep Modulus at 40°C = 10 MPa.

Application

Base material for TC50 and higher.

Construction Procedures

MIXING: In stationary plant, bitumen temperatures: To be determined for specific sources.

LAYING: By mechanical paver at minimum temperatures: To be determined for specific sources.

COMPACTION:
- Mean density: 95 % maximum theoretical density determined in the laboratory (no individual result should be below 93 %).
- Minimum temperature at end of compaction: To be determined for specific sources.
- Steel wheel roller (5000 - 7000 kg/m of roll width).
- Pneumatic tyred roller (Min. 4 tonnes per wheel).
- Vibratory rollers (Min. 200 kg/m of roll width).
- Minimum compacted layer thickness: 60 mm.
- Maximum compacted layer thickness: 100 mm.

8.9.7 Charts for Cobblestones, Concrete Block Paving, and Cement Concrete for Rigid Pavements

Materials Specification Charts	
PAVING BLOCKS	
Chart PB1	**Cobblestone Paving**

Materials Requirements						
Cobblestone, Kerbstones and Intersection Stones						
Cobblestone Dimension					**Qualities**	
Category	**Type**	**Length** (mm)	**Breadth** (mm)	**Depth** (mm)	**Compressive strength at 28 days cure** (MPa)	Min 25
Cubical Shape	10	100	100	100	**Water Absorption** (%)	Max 10
	11/13	110 – 130	110 – 130	130	**Los Angeles Abrasion** (%)	Max 50
	14/17	140 - 170	140 - 170	150	**Specific Gravity**	2 – 2.8
Stretcher Shape	13	≥ 130	110	110	**Tolerance**	
	16	≥ 160	110 – 130	150	Faces raw chiselled with max 5 mm roughness difference, crack-free and clean	
	20	≥ 200	140 – 160	150		

Kerbstone and Intersection Stone Dimension

- Kerbstones shall be rectangular blocks of 400 mm (length) x 100 mm (width) x 200 mm (depth) precast, or chiselled, or machine cut.
- The intersection stones shall comprise of the first two rows of cobblestone as specified above laid adjacent to the kerbstone.

Bedding and Joint Filler Material

This shall be naturally occurring clean sand or crushed rock fines, free from clay, lumps or other deleterious material with a grading curve falling within the following envelope:

Nominal Sieve Size (mm)	**Percentage by Mass Passing** (%)
10.00	100
5.00	90 – 100
2.36	75 – 100
1.18	55 – 90
600 µm	34 – 70
300 µm	8 – 35
150 µm	0 – 10
75 µm	0 – 3

Application

Base/Surfacing for low-speed areas with Traffic classes: TC0.25, TC0.5 and TC1 traffic.

Construction Procedures

- Sub-base/Foundation material shall be at least G30 in accordance with Chart GM8.
- Kerbs shall be set & embedded in concrete (class 15/20) to line/levels per design/drawings.
- After installation of kerbs, prime sub-base with MC 70 or MC 30 cutback bitumen depending on the permeability of the underlying material. Allow prime to cure/dry.
- Apply bedding sand on the sub-base to a thickness of 25 – 50 mm.
- Pack stones tightly, brush sand into gaps between stones, compact with a light non-vibratory roller or plate compactor.
- Brush in more sand, leave it on surface for 2 weeks after traffic opening, then brush off.

1 2 3 4 5 6 7 8 Pavement Materials

Materials Specification Charts

PAVING BLOCKS

Chart PB2	Concrete Block Paving Surfacing

Materials Requirements

Precast Concrete Blocks

These shall be manufactured in compliance to KS 827 and are graded in the following strength categories:

Grade	Nominal Thickness (mm)	Compressive Strength (N/mm^2)	Traffic Application
Heavy Duty (H)	80	49	Main roads & heavy industrial applications
	60	49	
Medium Duty (M)	60	35	Estate roads and parking areas
Light Duty (L)	50	25	Domestic driveways, parking areas & sidewalks

Categories of Paving Blocks by Dimension

Category	Length (mm)	Width (mm)	Thickness
Type R	200	100	As above
Type S	Any shape fitting within a 295mm square coordinating space, having a work size of not less than 80mm		

Gradings of Aggregates for Concrete Blocks

Sieve Size (mm)	% Passing
14.00	100
10.00	85 – 100
6.30	0 – 55
5.00	0 – 10

Bedding and Joint Filler Material

This shall be naturally occurring clean sand or crushed rock fines, free from clay, lumps or other deleterious material with a grading curve falling within the following envelope:

Nominal Sieve Size (mm)	Percentage by Mass Passing (%)
10.00	100
5.00	90 – 100
2.36	75 – 100
1.18	55 – 90
600 µm	34 – 70
300 µm	8 – 35
150 µm	0 – 10
75 µm	0 – 3

Application

PCB + sand layer as Base/Surfacing for low-speed traffic as indicated above.

Construction Procedures

- Sub-Base/Foundation material shall be at least G30 in accordance with Chart GM8.
- The kerbs shall be set in concrete class C15/20 to line and levels as per design drawings.
- After kerb installation, the sub-base shall be primed with MC 70 / MC 30 cutback bitumen depending on the permeability of the underlying material. Allow prime to cure/dry.
- Apply bedding sand on the sub-base to a thickness of 25 – 50 mm.
- Pack blocks tightly, brush sand into spaces between PCB, compact with a light non-vibratory roller or plate compactor.
- Brush more sand in. Leave loose sand on surfacing for 2 weeks after opening to traffic, then brush off.

Materials Specification Charts	
CEMENT CONCRETE PAVEMENT	
Chart CP1	**Cement Concrete (Class 25) (C25)**

Materials Requirements

Combined Aggregate Grading		Coarse Aggregate > 2 mm Requirements	
Sieve Size (mm)	**% by Weight Passing**	**Crushing Ratio** (%)	Min. 80
	0/9.5	**LAA Max.** (%)	35
37.5	100	**ACV Max.** (%)	28
28	90 - 100	**FI Max.** (%)	25
20	65 - 95	**Combined Aggregate Requirements**	
10	40 - 70	**Fines** (passing 0.425 mm)	Non-plastic
5	30 - 55	**Sand Equivalent**	Min. 30
2	18 - 40	**SSS Max.** (%)	Max. 12
1	13 - 32	**Organic Matter** (%)	Max. 0.3
0.425	9 - 24		
0.075	0 - 6		

Application

Pavement and surfacing on steep sections.

Construction Procedures

PROCEDURES:
- Minimum thickness of compacted layer: 100 mm, Tolerance ±10 mm.
- Mixing: In stationary concrete mixers
- Laying: by labour, within prefixed formwork or using a fixed form paver.

COMPACTION:
- Using poker vibrators or any other suitable approved method e.g., vibrating beams, roller screed.
- After compaction and after initial set, drag brush transversely across the surface to provide rough surface for traffic use.

PROTECTION AND CURING:
- Maximum time from mixing to final vibration 45 mins, unless proven otherwise through the action of retarding admixtures.
- 24hrs after construction, cover with sand and apply water/cure for 7 days.
- No traffic permitted for 7 days.
- Seal slab joints with emulsion slurry.

Cement

Portland cement (CEM I – 42.5 MPa) complying to KS EAS 18-1

Mix Design

Cube (150 mm) compressive strength after 28 days curing to be greater than 25 MPa.

Materials Specification Charts

CEMENT CONCRETE PAVEMENT

Chart CP2	JUC/JRC/CRCP Concrete (1-400 MCESA)

This is for general paving. Other specialised paving concretes are available e.g., salt water resistant or sulphate resistant. The maximum size of coarse aggregate (D) is usually 40 mm, but when the spacing between longitudinal reinforcement is less than 90 mm, D is 20 mm. Also, 20 mm is preferable to achieving for forming joints and a good surfacing.

Aggregate gradings are to be determined can be taken from KS 95 (see below) or can be determined during the mix design process and can vary according to the required strength, maximum aggregate size, method of laying, available materials, etc. See KS 95, KS 594, KS EAS 131-1, BS EN 206, BS EN 12620 and BS 8500 and Design of normal concrete mixes 2nd Edn, BRE Ltd). Producer to minimise risk of Alkali-Silica Reaction (ASR) and demonstrate this by Aggregate Reactivity tests.

Materials Requirements

All-in Aggregate Grading (KS 95)

Sieve Size (mm)	% by Weight Passing	
	D = 20 mm	D = 40 mm
37.5	100	95-100
20	95-100	45-80
5	35-55	25-50
0.6	10-35	8-30
0.15	0-8**	0-8*
0.075	0 - 4**	0 - 4**

** 10 for crushed rock sand.*

*** varies from 2 to 11 based on aggregate type.*

Coarse Aggregate > 2 mm Requirements (KS 1238)

Flakiness Index	≤50 uncrushed gravel ≤40 crushed rock/gravel
Max Shell Content	>10mm, Max. 8 % by mass
Oven dry: 10% Fines Value or	≥100
Max Agg. Impact Value	30 %
Max Acid Soluble Sulphate content	AS ≤ 0.8 % or 0.2 % by mass
Drying shrinkage	≤ 0.075 %
Los Angeles Abrasion	Max 50/40/35/30
Total Sulphur Content	S1 (≤1 %)

Combined Aggregate Requirements

Fines (passing 0.075 mm)	Non-plastic
Sulphates	Max 4 % by mass
Non-Concrete Products	Max 1 % by mass
Reinforced Concrete: Chloride Content	0.03, 0.05 or 0.2, 0.4, 1.0 % by mass of cement

Application

Pavement with concrete surfacing. CRCP can also be built with 30 mm asphalt surfacing.

Construction Procedures

PROCEDURES:
- Minimum thickness of compacted layer: 150 mm (URC/JRC), 200mm (CRCP). Tolerance ±6 mm.
- Slip membrane: for JUC and JRC it is required, for CRCP it is not required.
- Mixing: Usually using ready mixed concrete from plant.
- Laying: Usually with fixed form or slip-form paver but can be hand laid.

COMPACTION:
- Using poker vibrators, vibrating beams, roller screed or any other approved method.
- After compaction and initial set, add surface texture by dragging tined rake or brush transversely across the surface (or other approved method).

PROTECTION AND CURING:
- After compaction, cover with plastic sheeting/wet sand or apply curing compound/ bitumen emulsion spray or spray with water for min. 7 days. Tentage can protect from rain damage.
- No traffic permitted for 7 days
- For JUC and JRC, joints between slabs should be sealed with approved joint sealant.

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Cement
PAV2 from BS8500-1. Portland cement (CEM I), Portland -slag cement (CEM II/A-S & II/B-S), Blastfurnace cement (CEM III/A and III/B), Portland-fly ash cement CEM II/A-V and II/B-V) and Pozzolanic cement (CEM IV/A) complying to KS EAS 18-1.
Steel Reinforcement (for JRC and CRCP)
Longitudinal: 12- or 16-mm diameter. See RDM 3.5 for spacings. Transverse: 12 mm diameter at 600 mm spacing.
Mix Design
For JUC/JRC usually C40/50 concrete, but design can incorporate Mean Cube (150 mm) strength after 28 days = 30, 35, 40, 45, 50, 55, 60 MPa. For CRCP usually C40/50 concrete (or C32/40 if constructed with >30 mm asphalt surfacing). Design can incorporate Flexural strengths (at 28 day) of 4.5, 5.0, 5.5, 6.0 MPa.

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Pavement Materials

Materials Specification Charts

CEMENT CONCRETE PAVEMENT

Chart CP3	Continuously Reinforced Concrete Base (1-400 MCESA)

As CRCB is to be overlaid with at least 100 mm asphalt, the concrete can be of a lesser standard than a concrete pavement with a concrete surfacing e.g., JUC, JRC or CRCP. The maximum size of coarse aggregate (D) is usually 40 mm, but when the spacing between longitudinal reinforcement is less than 90 mm, D is 20 mm.

Aggregate gradings are to be determined can be taken from KS 95 (see below) or can be determined during the mix design process and can vary according to the required strength, maximum aggregate size, method of laying, available materials, etc. See KS 95, KS 594, KS EAS 131-1, BS EN 206, BS EN 12620 and BS 8500 and Design of normal concrete mixes 2nd Edn, BRE Ltd). Producer to minimise risk of Alkali-Silica Reaction (ASR) and demonstrate this by Aggregate Reactivity tests.

Materials Requirements

All-in Aggregate Grading (KS 95)		
Sieve Size (mm)	% by Weight Passing	
	D = 20 mm	D = 40 mm
37.5	100	95-100
20	95-100	45-80
5	35-55	25-50
0.6	10-35	8-30
0.15	0-8**	0-8*
0.075	0 - 4**	0 - 4**

** 10 for crushed rock sand. ** varies from 2 to 11 based on aggregate type.*

Coarse Aggregate > 2 mm Requirements (KS 1238)	
Flakiness Index	≤50 uncrushed gravel ≤40 crushed rock/gravel
Max Shell Content	>10mm, Max. 8 % by mass 5-10mm Max. 20 % by mass
Oven dry: 10% Fines Value or	≥100
Max Agg. Impact Value	30 %
Max Acid Soluble Sulphate Content	AS ≤ 0.8 % or 0.2 % by mass
Drying Shrinkage	≤ 0.075 %
Los Angeles Abrasion	Max 50/40/35/30
Total Sulphur Content	S1 (≤1 %)
Combined Aggregate Requirements	
Fines (passing 0.075 mm)	Max 50/40/35/30
Sulphates	Max 4 % by mass
Non-Concrete Products	Max 1 % by mass
Reinforced Concrete: Chloride Content	0.03, 0.05 or 0.2, 0.4, 1.0 % by mass of cement

Application

Main pavement layer. Also requires an asphalt surfacing (minimum thickness 100mm).

Construction Procedures

PROCEDURES:
- Minimum thickness of compacted layer: 150 mm (URC/JRC), 200mm (CRCP). Tolerance ±6 mm.
- No slip membrane required.
- Mixing: Usually using ready mixed concrete from plant.
- Laying: usually by fixed form or slip-form paver.

COMPACTION:
- Using poker vibrators or any other suitable approved method e.g., vibrating beams, roller screed.
- After compaction and after initial set, drag tined rake, brush etc, transversely across the surface to provide surface texture.

PROTECTION AND CURING:
- After compaction, apply a curing compound or a bitumen emulsion spray or wet sand or a mist/fog/light spray of water for at least 7 days. Tentage can protect from rain.
- No traffic permitted for 7 days.

Cement

PAV2 from BS8500-1. Portland cement (CEM I), Portland -slag cement (CEM II/A-S & II/B-S), Blastfurnace cement (CEM III/A and III/B), Portland-fly ash cement CEM II/A-V and II/B-V) and Pozzolanic cement (CEM IV/A) complying to KS EAS 18-1

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Steel Reinforcement (for JRC and CRCP)
Longitudinal: 12- or 16 mm diameter. See RDM 3.5 for spacings. Transverse: 12 mm diameter at 600mm spacing.
Mix Design
C32/40. Minimum Characteristic Cube (150 mm) strength after 28 days curing = 40 MPa. Flexural strength = 5 MPa.

Source: *UK DMRB, CD226, MCHW Vol 1 Series 1000 & NG1000, BS 882, BS EN 12620+A1, BS 8500, Design of normal concrete mixes 2nd Edn, BRE Ltd).*

Materials Specification Charts

CEMENT CONCRETE PAVEMENT

Chart CP4	Roller Compacted Concrete (RCC) (1-400 MCESA)

Aggregate gradings can be taken from MCHW (see below). See KS 95, KS 594, KS EAS 131-1, BS EN 206, BS EN 12620 and BS 8500 and Design of normal concrete mixes 2nd Edn, BRE Ltd). The concrete producer is to minimise risk of Alkali-Silica Reaction (ASR) and demonstrate this by Aggregate Reactivity tests.

The cement type, aggregate type/grading and water content used in RCC is different to that used in other types of concrete pavement in order to produce a drier, stiffer, zero slump concrete.

Materials Requirements

All-in Aggregate Grading			Coarse Aggregate > 2 mm Requirements	
Sieve Size (mm)	**% by Weight Passing**		**Flakiness Index**	≤50 uncrushed gravel
	0/14*	**0/20***	**Max Shell Content**	>10mm, Max. 8 % by mass
31.5	100	100	**Oven Dry: 10% Fines Value or**	≥100
20	100	90 - 100	**Max Agg. Impact Value**	30 %
14	86 - 100	78 - 94	**Max Acid Soluble Sulphate Content**	AS ≤ 0.8 % or 0.2 % by mass
10	72 - 95	62 - 86	**Drying Shrinkage**	≤ 0.075%
8	68 - 90	56 - 80	**Los Angeles Abrasion**	Max. 35 (LA_{35})
4	52 - 74	38 - 59	**Total Sulphur Content**	S1 (≤1%)
2	41 - 61	28 - 48	**Magnesium Sulphate**	MS18
1	30 - 50	19 - 39	**Combined Aggregate Requirements**	
0.5	20 - 37	15 - 31	**Fines** (passing 0.425 mm)	Non-plastic
0.25	11 - 25	9 - 23	**Sulphates**	Max 4% by mass
0.125	6 - 15	6 - 15	**Non-Concrete Products**	Max 1% by mass
0.063	2 - 10	2 - 10		
** The maximum coarse aggregate size of 20 mm is usually specified for RCC that is to be overlaid with asphalt. A maximum 14 mm can be specified if it is to have a concrete surface.*				

Application

RCC is the main pavement layer. An additional asphalt surfacing is usually added.

Construction Procedures

PROCEDURES:
- Laid in one layer. Minimum compacted layer thickness = 165 mm, Tolerance ±6 mm.
- Mixing: Usually using ready mixed concrete from plant.
- Laying: by slip-form paver. No formwork required.
- Applied finishes (e.g. brushed) are generally not possible. An asphalt surfacing (min 90 mm thick) is usually added for skid resistance and ride quality purposes.

COMPACTION and INDUCED CRACKING:
- After laying, rolling should be carried out by vibratory rollers followed by a PTR (Pneumatic Tyred Roller) if required and finished with a static roller for an even finish.
- Often, rolling alone does not create enough cracks. Additional transverse ‘joints’ should be created by grooving the fresh concrete to form straight vertical grooves (<20 mm wide, at 2.5 m centres +/-0.3m), to a depth of ¼ to 1/3 the slab thickness over the full width of the pavement. Bitumen emulsion is then poured/sprayed into the groove to form a crack-inducing membrane. During final compaction of the mixture, by rolling, the groove surface shall be fully closed along its full length.
- Longitudinal cracks shall also be induced, using the procedure specified above, under each lane line and the edge line between the nearside lane and a hard shoulder.
- Saw-cutting of hardened RCC as an alternative to induced cracking is NOT permitted.
- No trafficking within 7 days unless RCC achieves 20 MPa.

PROTECTION AND CURING:
- Apply curing compound asap after laying or if asphalt layer is to be added then a prime coat of bitumen emulsion can be used as the curing membrane for the RCC layer.

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Aggregate
Crushed gravel/recycled aggregates are not allowed as aggregate interlock is so important.
Cement
Portland cement (CEM I), Portland -slag cement (CEM II/A-S & II/B-S), Portland-fly ash cement CEM II/A-V and II/B-V) and Pozzolanic cement (CEM IV/A). Minimum cement/combination content = 270 kg/m3. Complying with KS EAS 18-1.
Steel Reinforcement (for JRC and CRCP)
Longitudinal: 12- or 16-mm diameter. See RDM 3.5 for spacings. Transverse: 12 mm diameter at 600mm spacing.
Mix Design
C40/50 concrete with minimum flexural strength = 5 MPa. i.e. Minimum characteristic cube (150 mm) strength after 28 days curing = 50 MPa.

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Appendices

Appendix A: Axle Loads and Equivalence Factors

Appendix A.1 Typical EFs for Class A Roads

Road	Statistic	B	MGV	HGV-1	HGV-2	OM	OH	OAT
Migori - Rongo	Mean EF	0.888	1.389	2.672	3.055	-	-	-
	EF Standard Deviation	0.784	2.719	5.545	4.191	-	-	-
	85th Percentile EF	1.756	2.004	6.972	9.405	-	-	-
Rongo - Migori	Mean EF	0.645	0.352	1.194	2.146	-	-	-
	EF Standard Deviation	0.878	0.991	1.53	2.873	-	-	-
	85th Percentile EF	1.119	0.541	2.251	4.117	-	-	-
Ahero - Oyugis	Mean EF	0.379	0.225	0.754	0.448	-	-	-
	EF Standard Deviation	0.551	0.929	2.196	0.206	-	-	-
	85th Percentile EF	0.660	0.177	0.289	0.628	-	-	-
Oyugis - Ahero	Mean EF	1.122	1.091	4.796	8.06	-	-	-
	EF Standard Deviation	2.512	3.071	7.177	11.384	-	-	-
	85th Percentile EF	1.512	1.699	11.334	22.608	-	-	-
Kisii - Oyugis	Mean EF	1.495	0.173	0.139	-	-	-	-
	EF Standard Deviation	2.674	0.676	0.227	-	-	-	-
	85th Percentile EF	3.47	0.075	0.209	-	-	-	-
Oyugis-Kisii	Mean EF	0.539	1.887	4.888	2.452	-	-	-
	EF Standard Deviation	0.940	2.696	5.264	3.304	-	-	-
	85th Percentile EF	1.099	4.66	7.126	5.587	-	-	-
Kisii-Rongo	Mean EF	0.930	0.764	1.147	4.965	-	-	-
	EF Standard Deviation	1.205	1.812	1.754	7.442	-	-	-
	85th Percentile EF	1.637	1.836	2.422	8.971	-	-	-
Rongo - Kisii	Mean EF	0.962	0.795	3.883	3.719	-	-	-
	EF Standard Deviation	0.978	1.959	5.177	7.835	-	-	-
	85th Percentile EF	1.602	1.647	7.691	7.58	-	-	-
Nairobi - Nakuru	Mean EF	0.931	0.538	1.258	1.361	0.439	1.258	1.771
	EF Standard Deviation	0.467	0.889	7.158	1.095	0.850	1.942	1.162
	85th Percentile EF	1.366	1.121	0.983	2.159	0.952	2.391	2.787
Nakuru - Nairobi	Mean EF	0.693	0.744	0.649	1.004	1.488	0.121	0.152
	EF Standard Deviation	0.464	1.18	1.369	2.009	2.548	0.177	0.360
	85th Percentile EF	1.083	1.086	1.09	1.759	2.923	0.232	0.094
Nairobi - Naivasha	Mean EF	4.15	2.064	2.624	8.828	-	-	-
	EF Standard Deviation	3.257	4.172	5.315	12.368	-	-	-
	85th Percentile EF	7.047	4.679	3.492	11.824	-	-	-
Naivasha - Nairobi	Mean EF	3.003	3.262	4.266	9.305	-	-	-
	EF Standard Deviation	1.847	7.957	8.448	40.593	-	-	-
	85th Percentile EF	5.094	5.767	6.596	11.418	-	-	-
Taveta - Voi	Mean EF	1.177	1.575	5.069	7.874	-	-	-
	EF Standard Deviation	1.854	3.81	7.834	12.732	-	-	-
	85th Percentile EF	1.748	3.549	11.86	13.47	-	-	-
Voi - Taveta	Mean EF	0.450	0.395	0.665	1.047	-	-	-
	EF Standard Deviation	0.439	0.929	1.209	1.922	-	-	-
	85th Percentile EF	0.903	0.680	2.309	1.62	-	-	-
Mombasa - Nairobi	Mean EF	1.156	1.964	4.292	5.524	-	-	-
	EF Standard Deviation	0.929	2.417	5.039	4.875	-	-	-
	85th Percentile EF	1.944	3.102	7.893	8.792	-	-	-
Nairobi - Mombasa	Mean EF	0.618	0.488	0.490	0.567	-	-	-
	EF Standard Deviation	0.440	1.211	0.799	0.981	-	-	-
	85th Percentile EF	0.986	0.823	1.118	1.114	-	-	-

Key:

B: *denotes Buses* ***MGV:*** *denotes Medium Goods Vehicles* ***HGV-1:*** *denotes Heavy Goods vehicles* ***HGV-2:*** *denotes Articulated Trucks* ***OM:*** *denotes Oil Tankers – Medium Vehicle* ***OH:*** *denotes Oil Tankers - Heavy Vehicle* ***OAT:*** *denotes Oil Tankers - Articulated Truck*

Appendix A: Axle Loads and Equivalence Factors

Appendix A.2 Typical EFs for Class B Roads

Road	Statistic	B	MGV	HGV-1	HGV-2	OM	OH	OAT
Busia - Kisumu	Mean EF	2.604	1.234	3.273	3.301	-	-	-
	EF Standard Deviation	1.565	3.197	4.469	8.967	-	-	-
	85^{th} Percentile EF	4.301	2.203	7.323	3.662	-	-	-
Kisumu - Busia	Mean EF	3.596	2.433	4.026	23.788	-	-	-
	EF Standard Deviation	2.099	4.007	7.485	19.967	-	-	-
	85^{th} Percentile EF	5.868	5.196	7.511	40.763	-	-	-
Kisumu - Mau Summit	Mean EF	4.911	2.646	4.669	8.893	-	-	-
	EF Standard Deviation	2.135	5.365	7.401	19.944	-	-	-
	85^{th} Percentile EF	7.074	5.822	8.494	15.996	-	-	-
Mau Summit - Kisumu	Mean EF	4.644	3.36	8.063	16.295	-	-	-
	EF Standard Deviation	2.422	6.056	12.569	20.578	-	-	-
	85^{th} Percentile EF	6.719	6.249	13.592	24.148	-	-	-
Mai Mahiu - Narok	Mean EF	2.875	1.381	4.519	8.816	-	-	-
	EF Standard Deviation	3.847	3.058	5.974	7.708	-	-	-
	85^{th} Percentile EF	6.572	2.769	7.727	18.461	-	-	-
Narok - Mai Mahiu	Mean EF	2.629	1.441	2.73	5.105	-	-	-
	EF Standard Deviation	4.023	3.383	4.657	6.054	-	-	-
	85^{th} Percentile EF	4.746	2.566	4.965	10.77	-	-	-
Kaplong - Narok	Mean EF	18.835	12.615	15.889	21.34	-	-	34.959
	EF Standard Deviation	18.159	15.076	18.277	18.008	-	-	16.323
	85^{th} Percentile EF	39.237	23.873	31.256	42.547	-	-	56.882
Narok - Kaplong	Mean EF	19.161	12.062	15.981	31.256	-	-	
	EF Standard Deviation	18.992	13.63	17.157	28.312	-	-	-
	85^{th} Percentile EF	40.684	20.791	34.212	56.313	-	-	-

Key:

B: *denotes Buses* ***MGV:*** *denotes Medium Goods Vehicles* ***HGV-1:*** *denotes Heavy Goods vehicles* ***HGV-2:*** *denotes Articulated Trucks* ***OM:*** *denotes Oil Tankers – Medium Vehicle* ***OH:*** *denotes Oil Tankers - Heavy Vehicle* ***OAT:*** *denotes Oil Tankers - Articulated Truck*

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Appendix A: Axle Loads and Equivalence Factors

Appendix A.3 Typical EFs for Class C Roads

Road	Statistic	B	MGV	HGV-1	HGV-2	OM	OH	OAT
Gilgil - Nyahururu	Mean EF	0.515	2.215	9.698	15.031	-	-	-
	EF Standard Deviation	1.135	4.914	9.697	28.461	-	-	-
	85th Percentile EF	0.713	5.056	20.578	20.148	-	-	-
Nyahururu - Gilgil	Mean EF	0.537	1.643	3.691	6.16	-	-	-
	EF Standard Deviation	0.639	4.449	7.492	10.61	-	-	-
	85th Percentile EF	0.888	2.535	9.178	13.765	-	-	-
Bondo - Kisian	Mean EF	1.893	0.220	0.651	0.538	-	-	-
	EF Standard Deviation	2.064	0.567	1.434	0.491	-	-	-
	85th Percentile EF	4.076	0.222	0.841	0.893	-	-	-
Kisian - Bondo	Mean EF	-	1.626	-	-	-	-	-
	EF Standard Deviation	-	2.018	-	-	-	-	-
	85th Percentile EF	-	2.972	-	-	-	-	-
Njabini - Olkalau	Mean EF	0.047	0.011	0.022	0.042	-	-	-
	EF Standard Deviation	0.318	0.049	0.046	0.088	-	-	-
	85th Percentile EF	0.022	0.006	0.025	0.042	-	-	-
Olkalau - Njabini	Mean EF	0.008	0.073	0.215	0.092	-	-	-
	EF Standard Deviation	0.022	0.217	0.256	0.122	-	-	-
	85th Percentile EF	0.015	0.079	0.425	0.187	-	-	-
Kaloleni-Mazeras	Mean EF	0.440	0.314	4.018	7.628	-	-	-
	EF Standard Deviation	2.471	0.891	5.296	10.904	-	-	-
	85th Percentile EF	0.541	0.442	10.031	22.269	-	-	-
Mazeras - Kaloleni	Mean EF	0.472	0.929	2.659	4.554	-	-	-
	EF Standard Deviation	0.361	1.907	3.522	7.387	-	-	-
	85th Percentile EF	0.867	1.805	4.939	9.473	-	-	-
Embakasi -Ruiru	Mean EF	-	1.855	2.223	6.133	-	-	-
	EF Standard Deviation	-	3.432	3.936	7.058	-	-	-
	85th Percentile EF	-	4.083	4.126	9.904	-	-	-
Ruiru - Embakasi	Mean EF	-	0.708	1.183	1.901	-	-	-
	EF Standard Deviation	-	1.205	1.383	3.048	-	-	-
	85th Percentile EF	-	1.453	2.407	3.658	-	-	-

Key:

B: *denotes Buses* ***MGV:*** *denotes Medium Goods Vehicles* ***HGV-1:*** *denotes Heavy Goods vehicles* ***HGV-2:*** *denotes Articulated Trucks* ***OM:*** *denotes Oil Tankers – Medium Vehicle* ***OH:*** *denotes Oil Tankers - Heavy Vehicle* ***OAT:*** *denotes Oil Tankers - Articulated Truck*

Appendix A: Axle Loads and Equivalence Factors

Appendix A.4 Typical EFs for Class D Roads

Road	Statistic	B	MGV	HGV-1	HGV-2	OM	OH	OAT
Nyeri - Kamakwa	Mean EF	0.709	0.167	6.422	-	-	-	-
	EF Standard Deviation	1.738	0.306	9.712	-	-	-	-
	85th Percentile EF	0.669	0.406	12.57	-	-	-	-
Kamakwa - Nyeri	Mean EF	0.745	0.396	3.433	-	-	-	-
	EF Standard Deviation	1.357	0.804	3.546	-	-	-	-
	85th Percentile EF	1.784	0.944	7.606	-	-	-	-
Nyeri - Ihururu	Mean EF	0.200	1.793	1.772	0.533	-	-	-
	EF Standard Deviation	0.366	9.933	2.307	0.252	-	-	-
	85th Percentile EF	0.519	0.145	4.72	0.658	-	-	-
Ihururu - Nyeri	Mean EF	0.682	0.231	1.226	0.378	-	-	-
	EF Standard Deviation	1.204	0.512	1.681		-	-	-
	85th Percentile EF	1.776	0.426	3.18	0.378	-	-	-
Kahuro - Muranga	Mean EF	0.193	0.215	1.354	2.079	-	-	-
	EF Standard Deviation	0.280	0.431	2.623	2.334	-	-	-
	85th Percentile EF	0.447	0.503	2.861	4.12	-	-	-
Muranga - Kahuro	Mean EF	0.173	0.321	1.464	0.234	-	-	-
	EF Standard Deviation	0.220	0.682	1.774	0.185	-	-	-
	85th Percentile EF	0.296	0.708	3.524	0.326	-	-	-
Kagio-Kerugoya	Mean EF	0.627	0.674	0.855	1.958	-	-	-
	EF Standard Deviation	0.590	2.363	1.206	1.889	-	-	-
	85th Percentile EF	1.036	1.044	1.854	4.017	-	-	-
Kerugoya - Kagio	Mean EF	0.206	0.150	0.120	0.136	-	-	-
	EF Standard Deviation	0.199	0.840	0.080	0.078	-	-	-
	85th Percentile EF	0.454	0.056	0.172	0.229	-	-	-
Molo - Olenguruone	Mean EF	0.454	2.584	1.111	2.215	-	-	-
	EF Standard Deviation	0.197	5.427	2.737	3.206	-	-	-
	85th Percentile EF	0.649	6.207	0.350	4.252	-	-	-
Olenguruone - Molo	Mean EF	0.316	0.385	3.336	2.421	-	-	-
	EF Standard Deviation	0.274	1.627	3.552		-	-	-
	85th Percentile EF	0.443	0.550	7.424	2.421	-	-	-
Isiolo - Ruiri	Mean EF	0.733	1.855	10.581	3.509	-	-	-
	EF Standard Deviation	0.960	1.862	7.989	4.762	-	-	-
	85th Percentile EF	1.707	3.549	17.976	5.866	-	-	-
Ruiri - Isiolo	Mean EF	0.205	0.508	0.909	0.688	-	-	-
	EF Standard Deviation	0.268	1.089	2.626		-	-	-
	85th Percentile EF	0.383	1.032	0.756	0.688	-	-	-

Key:

__B:__ denotes Buses __MGV:__ denotes Medium Goods Vehicles __HGV-1:__ denotes Heavy Goods vehicles __HGV-2:__ denotes Articulated Trucks __OM:__ denotes Oil Tankers – Medium Vehicle __OH:__ denotes Oil Tankers - Heavy Vehicle __OAT:__ denotes Oil Tankers - Articulated Truck

Appendix B: Refusal Density Testing (Vibrating Hammer Method)

Appendix B.1 Equipment

The equipment and the method of compaction used in the vibrating hammer test procedure for compacting AC 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 102 mm.

ii. A tamping foot with a diameter of 146 mm.

iii. 2 x shanks for the tamping feet.

iv. 8 x 152-153 mm 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.

Appendix B.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.

iii. Cores cut from the road.

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 baseplate, previously heated in the oven, is placed on top of the mould which is then turned over. The sample is driven to the new base plate with the hammer and large tamping foot. The compaction sequence is then repeated. The free base plate should be returned to the oven between compaction cycles.

Compaction of cores to refusal density

Pre-construction field trials, and subsequent monitoring for quality control purposes, will involve the compaction to refusal of 150 mm 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:

$$PRD = \frac{Bulk\ Density\ of\ Core}{Bulk\ Density\ after\ PRD\ compaction} \times 100$$

Equation B.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.

Appendix B.3 Refusal Density Design

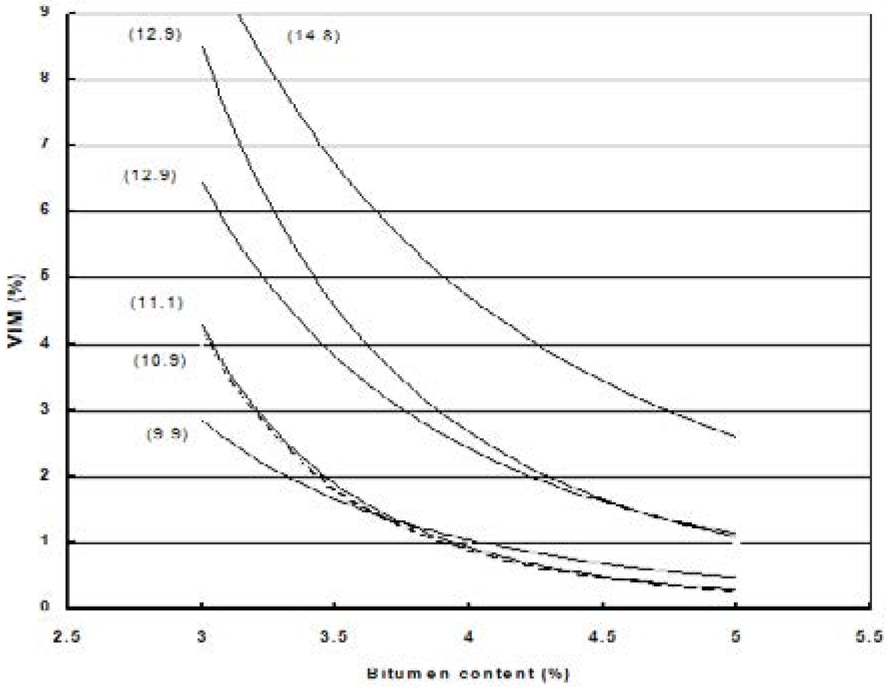
Firstly, a Marshall design should be carried out to ensure that the aggregate to be used in the production of AC for severe sites will meet the Marshall design requirements for very heavy traffic described in Chapter 8 of this manual.

Sometimes there is a choice of aggregate sources or sizes that are available for making AC. 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.

1 2 3 4 5 6 7 8 9 10 Appendices

Figure B.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, 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 % VIM in the Marshall test and then at decreasing increments of not more than 0.5 %. Tests at four bitumen contents should be sufficient to allow the bitumen content which gives 3 % 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 (AASHTO T 209/ASTM D2041) must also be determined 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 25 mm.

Whilst designing to refusal density will provide rut resistant mixes, experience may show that designing to 3 % 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 % 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.

Compaction Specifications for AC Designed to Refusal Density.

The relative level of compaction required in the constructed layer of AC 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 % of its refusal density and no individual value should be less than 93 % 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.

Durability of AC 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 %, VIM can be expected to range from 8 to 10 %. 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 37.5 mm. 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.

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 AC 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 AC has been constructed. Surface hardness tests 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) can also be used to surface the AC layer.

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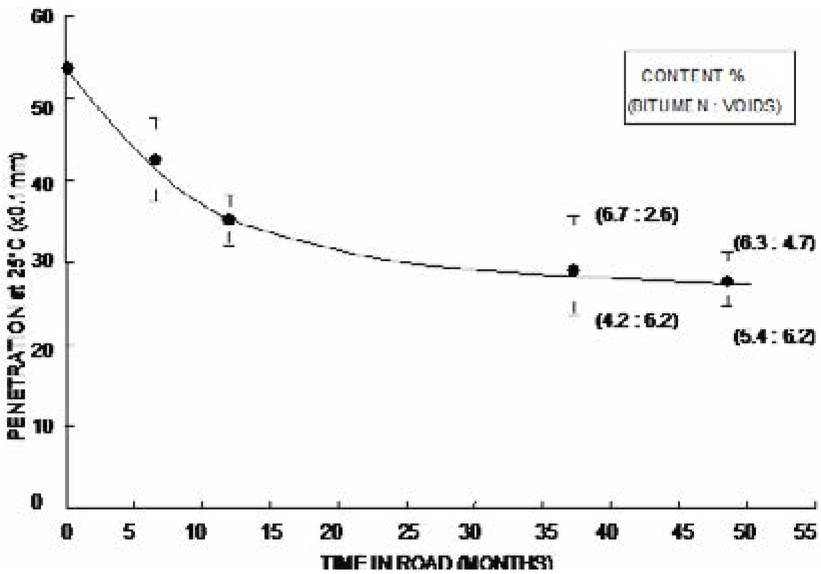
Figure B.2 Relationship Between Age and Bitumen Penetration for Surface Dressed Bitumen Macadam Base



Appendix B.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. They 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.

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 C: Bailey Aggregate Blending Method

Appendix C.1 Basic Principles

To develop a method for combining aggregates to optimise aggregate interlock and provide the proper volumetric properties, it is necessary to understand some of the controlling factors that affect the design and performance of these mixtures. The explanation of coarse and fine aggregates given in the following section provide a background for understanding the combination of aggregates. The Bailey Method builds on that understanding and provides more insight into the combination of aggregates for use in an asphalt mixture.

The Bailey Method uses two principles that are the basis of the relationship between aggregate gradation and mixture volumetrics:

- Aggregate packing.
- Definition of coarse and fine aggregate.

With these principles, the primary steps in the Bailey Method are:

- Combine aggregates by volume.
- Analyse the combined blend.

Appendix C.2 Aggregate Packing

Aggregate particles cannot be packed together to fill a volume completely. There will always be space between the aggregate particles. The degree of packing depends on:

- **Type and amount of compactive energy**. Several types of compactive force can be used, including static pressure, impact (e.g., Marshall hammer), or shearing (e.g., gyratory shear compactor or California kneading compactor). Higher density can be achieved by increasing the compactive effort (i.e., higher static pressure, more blows of the hammer, or more tamps or gyrations).
- **Shape of the particles**. Flat and elongated particles tend to resist packing in a dense configuration. Cubical particles tend to arrange in dense configurations.
- **Surface texture of the particles.** Particles with smooth textures will re-orient more easily into denser configurations. Particles with rough surfaces will resist sliding against one another.
- **Size distribution (gradation) of the particles**. Single-sized particles will not pack as densely as a mixture of particle sizes.
- **Strength of the particles**. Strength of the aggregate particles directly affects the amount of degradation that occurs in a compactor or under rollers. Softer aggregates typically degrade more than strong aggregates and allow denser aggregate packing to be achieved.

The properties listed above can be used to characterise both coarse and fine aggregates. The individual characteristics of a given aggregate, along with the amount used.

in the blend, has a direct impact on the resulting mix properties. When comparing different sources of comparably sized aggregates, the designer should consider these individual characteristics in addition to the Bailey Method principles presented. Even though an aggregate may have acceptable characteristics, it may not combine well with the other proposed aggregates for use in the design. The final combination of coarse and fine aggregates, and their corresponding individual properties, determines the packing characteristics of the overall blend for a given type and amount of compaction. Therefore, aggregate source selection is an important part of the asphalt mix design process.

Appendix C.3 Coarse and Fine Aggregate

The traditional definition of coarse aggregate is any particle that is retained by the 4.75 mm sieve. Fine aggregate is defined as any aggregate that passes the 4.75 mm sieve (sand, silt, and clay size material). The same sieve is used for 9.5 mm mixtures as 25.0 mm mixtures.

In the Bailey Method, the definition of coarse and fine is more specific in order to determine the packing and aggregate interlock provided by the combination of aggregates in various sized mixtures. The Bailey Method definitions are:

- **Coarse Aggregate:** Large aggregate particles that when placed in a unit volume create voids.
- **Fine Aggregate:** Aggregate particles that can fill the voids created by the coarse aggregate in the mixture.

From these definitions, more than a single aggregate size is needed to define coarse or fine. The definition of coarse and fine depends on the nominal maximum particle size (NMPS) of the mixture.

In a dense-graded blend of aggregate with a NMPS of 37.5 mm, the 37.5 mm particles come together to make voids. Those voids are large enough to be filled with 9.5 mm aggregate particles, making the 9.5 mm particles fine aggregate. Now consider a typical surface mix with a NMPS of 9.5 mm. In this blend of aggregates, the 9.5 mm particles are considered coarse aggregate.

In the Bailey Method, the sieve which defines coarse and fine aggregate is known as the primary control sieve (PCS), and the PCS is based on the NMPS of the aggregate blend. The PCS is defined as the closest sized sieve to the result of the PCS formula:

$$PCS = NMPS \times 0.22$$

Where,

PCS = for the overall blend

$NMPS$ = NMPS for the overall blend, which is one sieve larger than the first sieve that retains more than 10 % (as defined by Superpave terminology). The factor 0.22 is an average, any value between 0.18 and 0.28 is acceptable.

Appendix C.4 Combining Aggregates by Volume

All aggregate blends contain an amount and size of voids, which are a function of the packing characteristics of the blend. In combining aggregates we must first determine the amount and size of the voids created by the coarse aggregates and fill those voids with the appropriate amount of fine aggregate.

Mix design methods generally are based on volumetric analysis, but for simplicity, aggregates are combined on a weight basis. Most mix design methods correct the percent passing by weight to percent passing by volume when significant differences exist among the aggregate stockpiles. To evaluate the degree of aggregate interlock in a mixture the designer needs to evaluate a mixture based on volume.

To evaluate the volumetric combination of aggregates, additional information must be gathered. For each of the coarse aggregate stockpiles, the loose and rodded unit weights must be determined, and for each fine aggregate stockpile, the rodded unit weight must be determined.

These measurements provide the volumetric data at the specific void structure required to evaluate interlock properties.

Appendix C.5 Loose Unit Weight of Coarse Aggregate

The loose unit weight of an aggregate is the amount of aggregate that fills a unit volume without any compactive effort applied. This condition represents the beginning of coarse aggregate interlock (i.e., particle-to-particle contact) without any compactive effort applied.

The loose unit weight is determined on each coarse aggregate using the shovelling procedure outlined in AASHTO T-19: Unit Weight and Voids in Aggregate, which leaves the aggregate in a loose condition in the metal unit weight bucket. The loose unit weight (density in kg/m3) is calculated by dividing the weight of aggregate by the volume of the metal bucket. Using the aggregate bulk specific gravity and the loose unit weight, the volume of voids for this condition is also determined. This condition represents the volume of voids present when the particles are just into contact without any outside compactive effort being applied.

Appendix C.6 Rodded Unit Weight of Coarse Aggregate

The rodded unit weight of aggregate is the amount of aggregate that fills a unit volume with compactive effort applied. The compactive effort increases the particle to particle contact and decreases the volume of voids in the aggregate. The rodded unit weight is determined on each coarse aggregate using the rodding procedure outlined in AASHTO T-19: Unit Weight and Voids in Aggregate, which leaves the aggregate in a compacted condition in the metal unit weight bucket. The rodded unit weight (density in kg/m^3) is calculated by dividing the weight of aggregate by the volume of the metal bucket. Using the aggregate bulk specific gravity and the rodded unit weight, the volume of voids for this condition is also determined. This condition represents the volume of voids present when the particles are further into contact due to the compactive effort applied.

Appendix C.7 Chosen Unit Weight of Coarse Aggregate

The designer needs to select the interlock of coarse aggregate desired in their mix design. Therefore, they choose a unit weight of coarse aggregate, which establishes the volume of coarse aggregate in the aggregate blend and the degree of aggregate interlock.

In the Bailey Method, coarse graded is defined as mixtures which have a coarse aggregate skeleton. Fine-graded mixtures do not have enough coarse aggregate particles (i.e., larger than the PCS) to form a skeleton, and therefore the load is carried predominantly by the fine aggregate. To select a chosen unit weight the designer needs to decide if the mixture is to be coarse-graded or fine-graded. The loose unit weight is the lower limit of coarse aggregate interlock. Theoretically, it is the dividing line between fine-graded and coarse-graded mixtures. If the mix designer chooses a unit weight of coarse aggregate less than the loose unit weight, the coarse aggregate particles are spread apart and are not in a uniform particle-to-particle contact condition. Therefore, a fine aggregate skeleton is developed and properties for these blends are primarily related to the fine aggregate characteristics.

The rodded unit weight is generally considered to be the upper limit of coarse aggregate interlock for dense-graded mixtures. This value is typically near 110 % of the loose unit weight.

As the chosen unit weight approaches the rodded unit weight, the amount of compactive effort required for densification increases significantly, which can make a mixture difficult to construct in the field.

For dense-graded mixtures, the chosen unit weight is selected as a percentage of the loose unit weight of coarse aggregate. If the desire is to obtain some degree of coarse aggregate interlock (as with coarse-graded mixtures), the percentage used should range from 95 % to 105 % of the loose unit weight. For soft aggregates prone to degradation the chosen unit weight should be nearer to 105% of the loose unit weight (2). Values exceeding 105 % of the loose unit weight should be avoided due to the increased probability of aggregate degradation and increased difficulty with field compaction.

With fine-graded mixtures, the chosen unit weight should be less than 90% of the loose unit weight, to ensure the predominant skeleton is controlled by the fine aggregate structure.

For all dense-graded mixtures, it is recommended the designer should not use a chosen unit weight in the range of 90 % to 95 % of the loose unit weight. Mixtures designed in this range have a high probability of varying in and out of coarse aggregate interlock in the field with the tolerances generally allowed on the PCS.

It is normal for an aggregate blend to consolidate more than the selected chosen unit weight due to the lubricating effect of asphalt binder. Also, each coarse aggregate typically contains some amount of fine material when the unit weights are determined, which causes both unit weights (i.e., loose and rodded) to be slightly heavier than they would have been, had this material been removed by sieving prior to the test. Therefore, a chosen unit weight as low as 95% of the loose unit weight can often be used and still result in some degree of coarse aggregate interlock.

After selecting the desired chosen unit weight of the coarse aggregate, the amount of fine aggregate required to fill the corresponding VCA is determined.

Appendix C.8 Rodded Unit Weight of Fine Aggregate

For dense-graded mixtures, the voids created by the coarse aggregate at the chosen unit weight are filled with an equal volume of fine aggregate at the rodded unit weight condition. The rodded unit weight is used to ensure the fine aggregate structure is at or near its maximum strength. Rodded unit weight is determined on each fine aggregate stockpile as outlined in the rodding procedure in AASHTO T-19: Unit Weight and Voids in Aggregate, which leaves the aggregate in a compacted condition in the unit weight container. For most fine aggregates, which typically have a NMPS of 4.75 mm or less, a proctor mould, 100-mm diameter is used, which is a metal mould, approximately 0.9 litre in volume. The rodded unit weight (density in kg/m^3) is calculated by dividing the weight of the aggregate by the volume of the mould. In a dense-graded mixture, the rodded unit weight is always used to determine the appropriate amount of fine aggregate needed to fill the voids in the coarse aggregate at the chosen unit weight condition. A chosen unit weight is not selected. Note that the rodded unit weight is not determined for dust sized material, such as mineral filler (MF) or bag house fines.

Appendix C.9 Determining a Design Blend

The only additional information required other than that typically used in a dense-graded mix design is the corresponding unit weight for each coarse and fine aggregate [excluding MF, bag house fines, and recycled asphalt pavement (RAP)]. The following decisions are made by the designer and used to determine the individual aggregate percentages by weight and the resulting combined blend:

- Bulk specific gravity of each aggregate.
- Chosen unit weight of the coarse aggregates.
- Rodded unit weight of the fine aggregates.
- Blend by volume of the coarse aggregates totalling 100 %.
- Blend by volume of fine aggregates totalling 100 %.
- Amount of 0.075 mm material desired in the combined blend if MF or bag house fines are being used.

The following steps are presented to provide a general sense of blending aggregates by volume.

1. Pick a chosen unit weight for the coarse aggregates, kg/m^3.
2. Calculate the volume of voids in the coarse aggregates at the chosen unit weight.
3. Determine the amount of fine aggregate to fill this volume using the fine aggregates rodded unit weight, kg/m^3.
4. Using the weight (density) in kg/m^3 of each aggregate, determine the total weight and convert to individual aggregate blend percentages.
5. Correct the coarse aggregates for the amount of fine aggregate they contain and the fine aggregates for the amount of coarse aggregate they contain, in order to maintain the desired blend by volume of coarse and fine aggregate.
6. Determine the adjusted blend percentages of each aggregate by weight.
7. If MF or bag house fines are to be used, adjust the fine aggregate percentages by the desired amount of fines to maintain the desired blend by volume of coarse and fine aggregate.
8. Determine the revised individual aggregate percentages by weight for use in calculating the combined blend.

Appendix C.10 Analysis of the Design Blend

After the combined gradation by weight is determined, the aggregate packing is analysed further. The combined blend is broken down into three distinct portions, and each portion is evaluated individually. The coarse portion of the combined blend is from the largest particle to the PCS. These particles are considered the coarse aggregates of the blend.

The fine aggregate is broken down and evaluated as two portions. To determine where to split the fine aggregate, the same 0.22 factor used on the entire gradation is applied to the PCS to determine a secondary control sieve (SCS). The SCS then becomes the break between coarse sand and fine sand. The fine sand is further evaluated by determining the tertiary control sieve (TCS), which is determined by multiplying the SCS by the 0.22 factor.

An analysis is done using ratios that evaluate packing within each of the three portions of the combined aggregate gradation. Three ratios are defined: Coarse Aggregate Ratio (CA Ratio), Fine Aggregate Coarse Ratio (FAc Ratio), and Fine Aggregate Fine Ratio (FAf Ratio).

These ratios characterise packing of the aggregates. By changing gradation within each portion modifications can be made to the volumetric properties, construction characteristics, or performance characteristics of the asphalt mixture.

Appendix C.11 CA Ratio

The CA Ratio is used to evaluate packing of the coarse portion of the aggregate gradation and to analyse the resulting void structure. Understanding the packing of coarse aggregate requires the introduction of the half sieve. The half sieve is defined as one half the NMPS. Particles smaller than the half sieve are called 'interceptors.' Interceptors are too large to fit in the voids created by the larger coarse aggregate particles and hence spread them apart. The balance of these particles can be used to adjust the mixture’s volumetric properties. By changing the quantity of interceptors it is possible to change the VMA in the mixture to produce a balanced coarse aggregate structure. With a balanced aggregate structure the mixture should be easy to compact in the field and should adequately perform under load. The equation for the calculation of the coarse aggregate ratio is given by:

$$CA\ Ratio = \frac{\%\ Passing\ Half\ Sieve - \%\ Passing\ PCS}{100\% - \%\ Passing\ Half\ Sieve}$$

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The packing of the coarse aggregate fraction, observed through the CA Ratio, is a primary factor in the constructability of the mixture. As the CA Ratio decreases (below ~1.0), compaction of the fine aggregate fraction increases because there are fewer interceptors to limit compaction of the larger coarse aggregate particles. Therefore, a mixture with a low CA Ratio typically requires a stronger fine aggregate structure to meet the required volumetric properties. Also, a CA Ratio below the corresponding range suggested in Table F1-1 could indicate a blend that may be prone to segregation. It is generally accepted that gap-graded mixes, which tend to have CA Ratios below these suggested ranges, have a greater tendency to segregate than mixes that contain a more continuous gradation. As the CA Ratio increases towards 1.0, VMA will increase. However, as this value approaches 1.0, the coarse aggregate fraction becomes 'unbalanced' because the interceptor size aggregates are attempting to control the coarse aggregate skeleton. Although this blend may not be as prone to segregation, it contains such a large quantity of interceptors that the coarse aggregate fraction causes the portion above the PCS to be less continuous. The resulting mixture can be difficult to compact in the field and have a tendency to move under the rollers because it does not want to 'lock up.' Generally, mixes with high CA Ratios have a S-shaped gradation curve in this area of the 0.45-power grading chart. Superpave mixtures of this type have developed a reputation for being difficult to compact.

As the CA Ratio exceeds a value of 1.0, the interceptor-sized particles begin to dominate the formation of the coarse aggregate skeleton. The coarse portion of the coarse aggregate is then considered 'pluggers,' as these aggregates do not control the aggregate skeleton, but rather float in a matrix of finer coarse aggregate particles.

Table C.1 Recommended Ranges of Aggregate Ratios

For dense-graded mixes e.g. AC						
Ratios	**NMAs** (mm)					
	37.5	**25.0**	**19.0**	**12.5**	**9.5**	**4.75**
CA Ratio	0.80 - 0.95	0.70 - 0.85	0.60 - 0.75	0.50 - 0.65	0.40 - 0.55	0.30 - 0.45
FAc Ratio	0.35 - 0.50	0.35 - 0.50	0.35 - 0.50	0.35 - 0.50	0.35 - 0.50	0.35 - 0.50
FAf Ratio	0.35 - 0.50	0.35 - 0.50	0.35 - 0.50	0.35 - 0.50	0.35 - 0.50	0.35 - 0.50
For Stone Mastic Asphalt (SMA)						
Ratios	**NMAs** (mm)					
	37.5	**25.0**	**19.0**	**12.5**	**9.5**	**4.75**
CA Ratio	-	-	0.35-0.50	0.25-0.40	0.15-0.30	-
FAc Ratio	-	-	0.60-0.85	0.60-0.85	0.60-0.85	-
FAf Ratio	-	-	0.65-0.90	0.65-0.90	0.65-0.90	-

Note: *FAc = fine aggregate coarse, Faf = fine aggregate fine. These ranges provide a starting point where no prior experience exists for a given set of aggregates. If the designer has acceptable existing designs, they should be evaluated to determine a narrower range to target for future designs.*

Appendix C.12 Coarse Portion of Fine Aggregate

All of the fine aggregate (i.e., below the PCS) can be viewed as a blend by itself that contains a coarse and a fine portion and can be evaluated in a manner similar to the overall blend. The coarse portion of the fine aggregate creates voids that will be filled with the fine portion of the fine aggregate. As with the coarse aggregate, it is desired to fill these voids with the appropriate volume of the fine portion of the fine aggregate without overfilling the voids. The equation that describes the fine aggregate coarse ratio (FAc) is given in Equation 3. As this ratio increases, the fine aggregate (i.e., below the PCS) packs together tighter. This increase in packing is due to the increase in volume of the fine portion of fine aggregate. It is generally desirable to have this ratio less than 0.50, as higher values generally indicate an excessive amount of the fine portion of the fine aggregate is included in the mixture. A FAc Ratio higher than 0.50, which is created by an excessive amount of natural sand and/or an excessively fine natural sand should be avoided. This type of a blend normally shows a 'hump' in the sand portion of the gradation curve of a 0.45 gradation chart, which is generally accepted as an indication of a potentially tender mixture. The equation for the calculation of the FAc Ratio is given by:

$$FAc = \frac{\%\ Passing\ SCS}{\%\ Passing\ PCS}$$

If the FAc Ratio becomes lower than the range of values in Table F1-1, the gradation is not uniform. These mixtures are generally gap-graded and have a 'belly' in the 0.45-power grading chart, which can indicate instability and may lead to compaction problems. This ratio has a considerable impact on the VMA of a mixture due to the blending of sands and the creation of voids in the fine aggregate. The VMA in the mixture will increase with a decrease in this ratio.

Appendix C.13 Fine Portion of Fine Aggregate

The fine portion of the fine aggregate fills the voids created by the coarse portion of the fine aggregate. This ratio shows how the fine portion of the fine aggregate packs together. One more sieve is needed to calculate the FAf, the TCS. The TCS is defined as the closest sieve to 0.22 times the SCS. The equation for the FAf Ratio is given by:

$$FAf = \frac{\%\ Passing\ TCS}{\%\ Passing\ SCS}$$

The FAf Ratio is used to evaluate the packing characteristics of the smallest portion of the aggregate blend. Similar to the FAc Ratio, the value of the FAf Ratio should be less than 0.50 for typical dense-graded mixtures. VMA in the mixture will increase with a decrease in this ratio.

Appendix C.14 Summary of Ratios

CA Ratio – This ratio describes how the coarse aggregate particles pack together and, consequently, how these particles compact the fine aggregate portion of the aggregate blend that fills the voids created by the coarse aggregate.

FAc Ratio – This ratio describes how the coarse portion of the fine aggregate packs together and, consequently, how these particles compact the material that fills the voids it creates.

FAf Ratio – This ratio describes how the fine portion of the fine aggregate packs together. It also influences the voids that will remain in the overall fine aggregate portion of the blend because it represents the particles that fill the smallest voids created.

These ratios are valuable for evaluating and adjusting VMA. Once an initial trial gradation is evaluated in the laboratory, other gradations can be evaluated on paper to choose a second trial that will have an increased or decreased VMA as desired. When doing the paper analysis, the designer must remember that changes in particle shape, strength and texture must be considered as well. The ratios are calculated from the control sieves of an asphalt mixture, which are tied to the NMPS. Table C-2 provides the listing of control sieves for various asphalt mixture sizes (both dense-graded mixes and stone mastic asphalt). The values in determining the aggregate ratios are the percent passing the control sieves for the final combined blend. The recommended range for the ratios is shown in Table C-2.

Table C.2 Recommended Ranges of Aggregate Ratios

Control Sieve	NMAs (mm)					
	37.5	25.0	19.0	12.5	9.5	4.75
Half Sieve	19.0	12.5	9.5	**	4.75	2.36
PCS	9.5	4.75	4.75	2.36	2.36	1.18
SCS	2.36	1.18	1.18	0.60	0.60	0.30
TCS	0.60	0.30	0.30	0.150	0.150	0.075

Note: *** The nearest 'typical' half sieve for a 12.5-mm NMPS mixture is the 4.75 mm. However, the 6.25 mm sieve actually serves as the breakpoint. Interpolating the percent passing value for the 6.25-mm sieve for use in the CA Ratio will provide a more representative ratio value.*

Appendix D: A Summary of the Marshall and Modified Marshall Asphalt Mix Design

Appendix D.1 General

Mix design for asphalt materials using the Marshall Method shall be based on the recommendations given in the Asphalt Institute Manual Series, MS-2. The standard Marshall method is suitable for the design and field control of AC mixtures containing aggregates with a maximum size of up to 25 mm. For aggregates with maximum size of 37.5 mm, the 152.4 mm mould should be used. The main steps involve:

1. Selection and blending of aggregates to achieve the particle size distributions shown the materials specification charts (BB1, SU8-SU10) in section 8.9. This often requires blending of 4 or 5 aggregate fractions.
2. Selection of the appropriate grade of bitumen (see materials specification charts BB1, SU8-SU10 in section 8.9).
3. Estimation of the design bitumen content.
4. Mixing of the aggregate and bitumen for at least 5 bitumen contents to obtain several and compaction at the appropriate level representing the design traffic.
5. Determination of volumetric properties bulk specific gravity (BSG), voids in mineral aggregate (VMA) and voids filled with bitumen (VFB), voids in the mix (VIM), stability, and flow (specification charts BB1, SU8-SU10) in section 8.9).
6. Check of the minimum theoretical film thickness.
7. Checking for the water sensitivity through modified Lottman test (AASHTO T283). The ITSwet/ITSdry ratio should be 0.8 or higher.
8. Conducting other performance tests (wheel tracking and fatigue testing).
9. Reformulation of the mix until the design mix requirements (see materials specification charts (BB1, SU8-SU10) in section 8.9) have been achieved.

Appendix D.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 25 kg 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 AC taken from an asphalt plant it is important to complete the Marshall compaction before the samples have cooled below the recommended compaction temperature.

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.

It is important that the sieve sizes used for the sieve analysis of the aggregates are the same as those specified in the final mix gradation.

Design of Aggregate Grading

Using the results of the sieve analysis obtained for each source of aggregate a blend is computed

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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.

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.

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 °C 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.

Appendix D.3 Preparation of Test Samples

Mass of Aggregate Required

The amount of aggregate required for each sample is that which will be sufficient to make compacted specimens 63.5 ± 1.27 mm high. This is normally approximately 1.2 kg and should be confirmed by compacting a trial sample of 1.2 kg 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:

$$Adjusted\ Weight = \frac{63.5 * Weight\ of\ Aggregate\ Used}{Specimen\ Height\ (mm)\ Obtained}$$

Equation D.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.

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:

$$DBC = 0.035\,a + 0.04b + Kc + F$$

Equation D.2

Where,

DBC	= approximate design bitumen content, per cent by weight of mix
a	= per cent of mineral aggregate retained on the 2.36 mm sieve
b	= per cent of mineral aggregate passing the 2.36 mm sieve and retained on the 0.075 mm sieve
c	= per cent of mineral aggregate passing the 0.075 mm sieve
K	= 0.15 for 11-15 % passing the 0.075 mm sieve 0.18 for 6-10 % passing the 0.075 mm sieve 0.20 for 5 % or less passing the 0.075 mm 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.

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.

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. 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.

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Appendix D.4 Testing of Specimens

Bulk Specific Gravity Determination

The bulk specific gravity is determined for each briquette at 25 °C in accordance with the test procedure described in AASHTO T 166 (ASTM D2726).

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 °C 5 °C. Each pair of briquettes is combined to give bulk samples to be tested in accordance with the AASHTO T 269 (ASTM D2041) procedure for the determination of maximum specific gravity of the mixes.

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.

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.

Appendix D.5 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 D.1.

Table D.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	-	-

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:

$$Gsb = \frac{P_1 + P_2 + \ldots.\ P_n}{\frac{P_1}{G_1} + \frac{P_2}{G_2} + \ldots.\ \frac{P_n}{G_n}}$$

Equation D.3

Where,

Gsb = bulk specific gravity for the total aggregate

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

$G_1, G_2 .. Gn$ = individual bulk specific gravities of aggregates

During the production of AC 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 AC, the BSG of the compacted material (G_{mb}) and the SG of the bitumen (G_b) used in the mix.

Appendix D.6 Calculation of Volumetric Properties of Individual Components

Effective Specific Gravity of Aggregate

When based on the Gmm of a bituminous mixture, the effective SG of the aggregate, Gse, includes all void spaces within the aggregate particles, except those that absorb bitumen, and is determined using:

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

Equation D.4

Where,

Gse = effective specific gravity of aggregate

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

P_b = bitumen content at which AASHTO T209 (ASTM D2041) test (G_{mm}) was performed, percent by total weight of mixture

Gb = specific gravity of bitumen

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. Gmm 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:

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

Equation D.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

Bitumen Absorption

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

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

Equation D.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

Effective Bitumen Content of the Mix

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

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

Equation D.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

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:

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

Equation D.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

In order to achieve a balance of mix properties it is important that the aggregate structure of an AC has sufficient VMA. The minimum VMA required is related to the nominal aggregate size as shown in Table D.2.

Table D.2 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

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:

$$VMA = 100\left(\frac{G_{mm} - G_{mb}}{G_{mm}}\right)$$

Equation D.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

Marshall criteria are as shown in Table D.3.

Table D.3 AC Wearing Course Specifications

Category and design traffic	No. of blows of Marshall compaction hammer	VFB (%)	VIM at Optimum Bitument Content (%)
Heavy	75[1]	65 - 75	3 - 5
Medium	50[2]	65 - 78	3 - 5
Light	35	70 - 80	3 - 5

Notes: 1: *For the 152.4 mm diameter mould, the number of blows = 112* ***2:*** *For the 152.4 mm diameter mould, the number of blows = 75*

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:

$$VMA = 100\left(\frac{VMA - VIM}{VMA}\right)$$

Equation D.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

Appendix D.7 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.

Appendix D.8 Selection of the Mix Design

The selection is aided by plotting graphs of the various design specification criteria against the bitumen contents used in the mix design test (Figure D1), and then developing a diagram of the volumetric properties of each mix and the bitumen content range in which they are achieved. An example of the diagram is given in Table D.4. The key driver is the VIM in the median of the specification range (usually 4 %). Then VMA, VFB, stability and flow are checked for the compliance with the specified design criteria.

Figure D.1 An Example of Volumetric Plots at Various Bitumen Contents

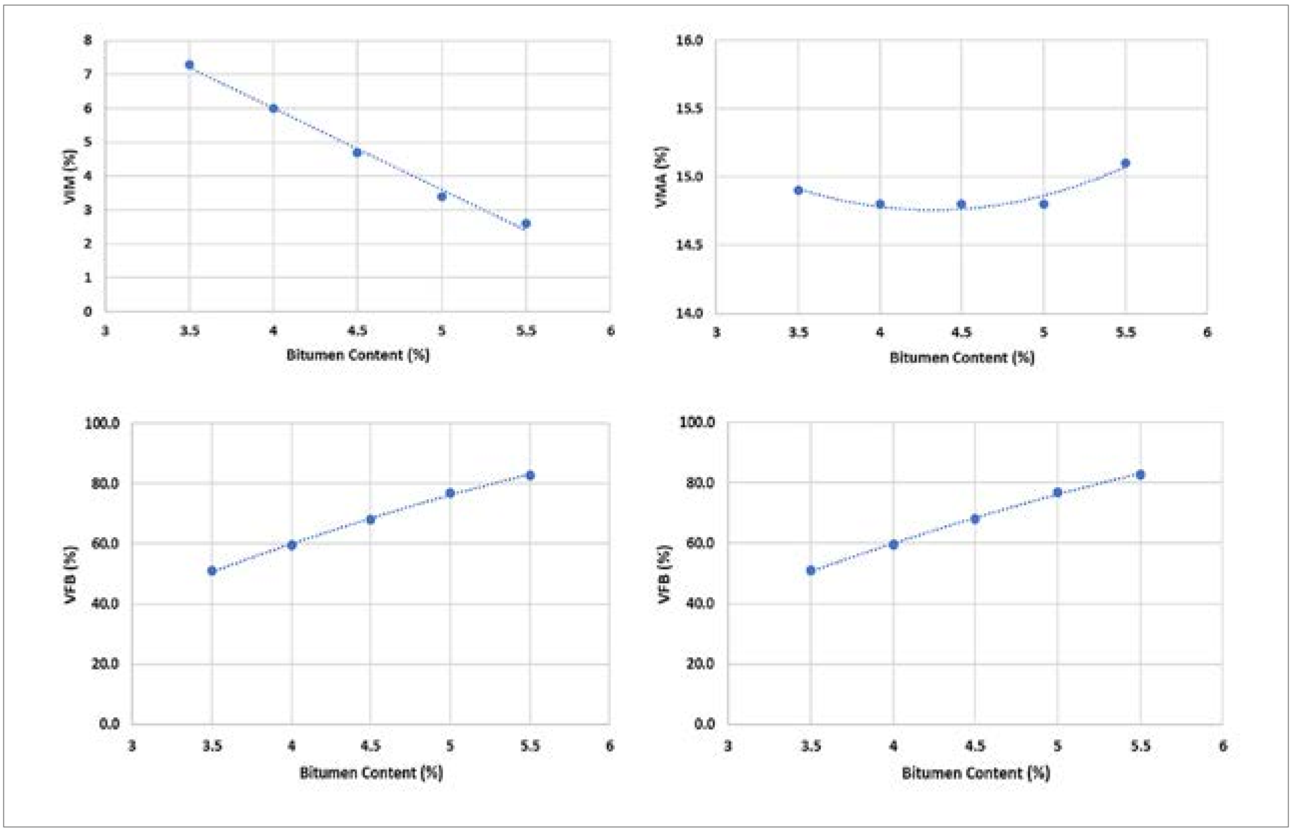
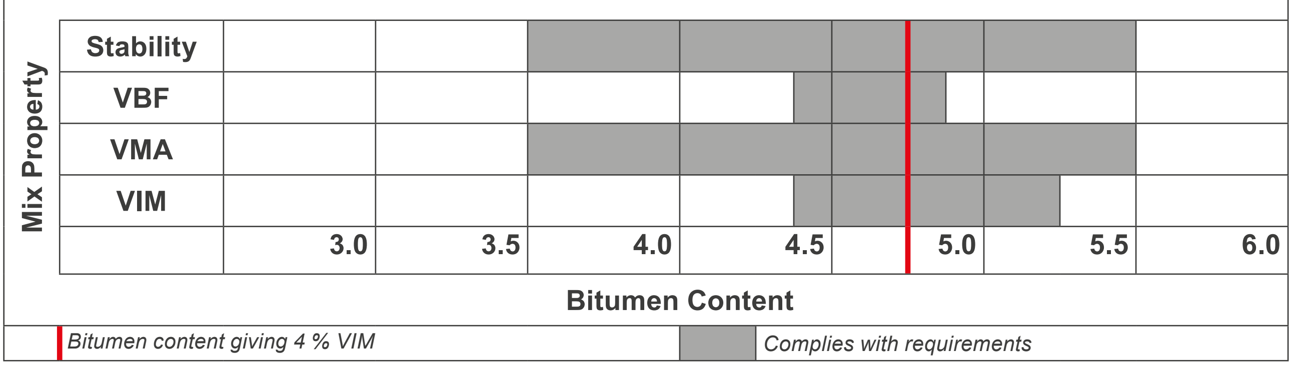


Table D.4 An Example of Acceptable Bitumen Range Complying with Design Criteria

Mix Property	MS-2 Criteria (Example Only)	% Range of Bitumen Content Giving Compliance with MS-2 Criteria (Example Only)
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



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Appendix D.9 Bitumen Film Thickness

Bitumen film thickness can be estimated using the following formula:

$$F = \frac{B_e}{100 - B} * \frac{1}{A} * \frac{1}{S} *10^6$$

Equation D.11

Where,

F = Film thickness

B_e = Effective bitumen content of AC (% by mass of mix)

B = Total bitumen content of AC (% 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:

$$(2 + 0.02a + 0.04b + 0.08c + 0.14d + 0.3e + 0.6f + 1.6g)*0.20482$$

Equation D.12

Where,

a = percentage passing 4.75 mm sieve

b = percentage passing 2.36 mm sieve

c = percentage passing 1.18 mm sieve

d = percentage passing 0.600 mm sieve

e = percentage passing 0.300 mm sieve

f = percentage passing 0.150 mm sieve

g = percentage passing 0.075 mm sieve

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.

Appendix D.10 Water Sensitivity Assessment

The water sensitivity of the selected mix should be checked through modified Lottman test (AASHTO T 283). The ITSwet/ITSdry ratio should be 0.8 or higher.

Appendix D.11 Modified Marshall Mix Design

Mixes for modified Marshall design must first meet all the requirements of the Marshall Mix described above. In addition to that, the critical modified steps involves the requirement to retain air voids of 3 % - 5 % after refusal compaction. This procedure is detailed in Appendix B. In Kenya, to achieve this, the particle size distribution of the aggregates used conforms to that of Superpave mixes. The Bailey aggregate blending method described in Appendix C may also be used to select the appropriate aggregate grading.

Appendix D.12 Conducting Other Performance Tests

For design traffic class TC80, selection of the design mix should be enhanced by undertaking a wheel tracking test (AASHTO T 324) to verify the rut resistance of the mix and the four-point bending beam test (AASHTO T 321) to guard against fatigue cracking.

Appendix E: A Summary of the Superpave Asphalt Mix Design

Appendix E.1 General

Mix design for asphalt materials using the Superpave Mix Design Method shall be based on the recommendations given in the Asphalt Institute Manual Series, MS-2. The standard Superpave mix design method is suitable for the design and field control of AC mixtures containing aggregates with a maximum size of up to 37.5 mm. The main steps involve:

1. Determination of the high and low pavement temperatures for design and selecting bitumen and aggregate materials that meet the design criteria.
2. Developing an aggregate blend that will meet the Superpave requirements.
3. Mixing and short-term aging the selected binder and aggregate blend.
4. Compacting specimens utilising the Superpave gyratory compactor according to design traffic levels.
5. Analysing the volumetric properties of the mixture.
6. Selecting the best aggregate and asphalt blend that meets the specified design criteria for the mixture.
7. Performance testing of the mixture for moisture sensitivity.
8. Conducting other performance tests (wheel tracking and fatigue testing).

Appendix E.2 Materials for Superpave

Pavement Temperature and 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 20 mm below the road surface whilst the low temperature is determined for the surface of the road. These are obtained from pavement temperature maps, or correlations of air temperature to pavement temperature, or site-specific studies. 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. Final selection of the grade of bitumen then takes into account the actual designations of standard grades of binders as given in Asphalt Institute Manual Series MS-26.

For example, the bitumen grade required for a high design temperature of 52 °C and a low design temperature of -10 °C is designated PG52-10. 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.

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 3 Pas 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):
    a. Maximum dynamic shear at a temperature appropriate to the road site.
    b. Physical hardening, tests on beams of bitumen.
    c. Creep stiffness criteria.
    d. 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.

The target design binder content can be estimated from experience or as per the Marshall Method (Equation D.2).

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.

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 E.1 and E.2.

Aggregate and binder specific gravities must be measured as specified in Table D.1, Appendix D.

Appendix E.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 600 kPa 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 1.16 degrees.

Table E.1 Particle Size Distribution for Superpave AC Wearing Courses

Sieve Size (mm)	% 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 E.2 Particle Size Distribution for Superpave AC Roadbase and Binder Courses

Sieve Size (mm)	% 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 E.3. Other design requirements are also given in Table E.4.

Table E.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, Manual Series No. 2 (MS-2) for other conditions.*

Table E.4 Superpave AC Design Requirements

<table>
<tr><th rowspan="3">Design traffic (esa x 10^6)[1]</th><th colspan="3">Required Relative Density (% of theoretical maximum specific gravity)</th><th colspan="5">Minimum Voids in Mineral Aggregate (VMA) (%)</th><th rowspan="3">Range of Voids Filled with Bitumen (%)</th><th rowspan="3">Range of Filler Binder Ratio</th></tr>
<tr><th colspan="3"></th><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>

Notes:

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 37.5 mm nominal maximum aggregate size mixtures, the specified lower limit of the VFB shall be 64 % for all design traffic levels.

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

4. For 9.5 mm nominal maximum aggregate size mixtures, the specified VFB range shall be 73 to 76 % for design traffic levels > 3 million esa.

5. If the aggregate gradation passes beneath the boundaries of the restricted zone specified in Tables E.1 or E.2, the filler to bitumen ratio range may be increased from 0.6 – 1.2 to 0.8 –1.6.

Appendix E.4 Volumetric Analysis

The samples are compacted to the appropriate number of gyrations selected from Table E.3. 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 its bulk specific gravity (AASHTO T 166/ASTM D 2726) and maximum specific gravity (Gmm) AASHTO T 209/ASTM D 2041) are determined.

Note that the volumetric properties are computed as per the Marshall Method described in Appendix D.

Appendix E.5 Selecting the Design Mix

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 % VIM at Ndesign. The criteria which must be met at this bitumen content are summarised in Table E.4. The selection is aided by plotting graphs of the various design specification criteria against the bitumen contents used in the mix design test (Figure E.1), and then developing a diagram of the volumetric properties of each mix and the bitumen content range in which they are achieved. An example of the diagram is given in Table E.5. The key driver is the VIM in the median of the specification range (usually 4 %). Then VMA, VFB, stability and flow are checked for the compliance with the specified design criteria.

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Appendices

Figure E.1 An Example (Superpave) of Volumetric Plots at Various Bitumen Contents

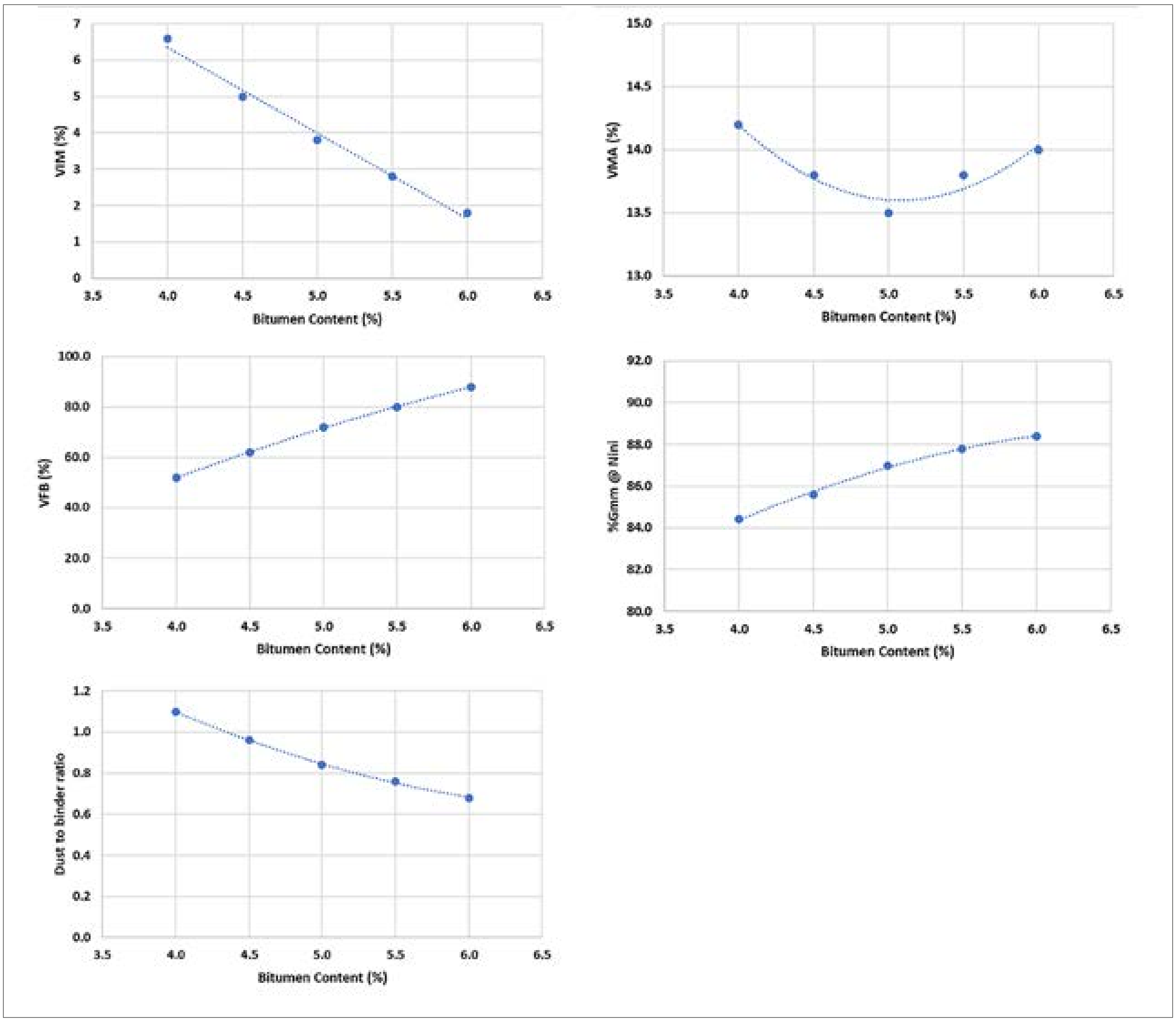
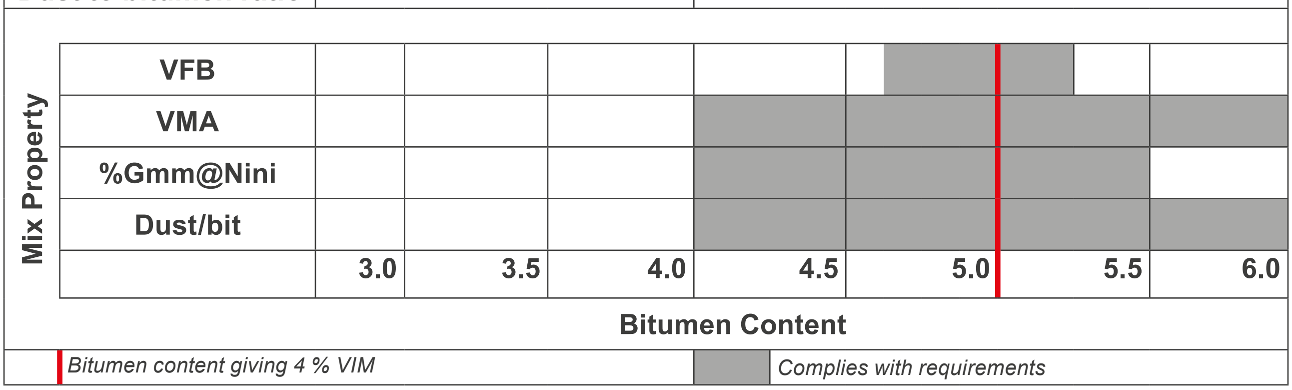


Table E.5 An Example (Superpave) of Acceptable Bitumen Range Complying with Design Criteria

Mix Property	MS-2 Criteria (Example Only)	% Range of Bitumen Content Giving Compliance with MS-2 Criteria (Example Only)
VIM	4 %	5.0
VFB	65 – 75 %	4.7 – 5.3
VMA	13% minimum	4.0 – 6.0
% Gmm	≤ 89	4.0 – 6.0
Dust to bitumen ratio	0.6 – 1.2	4.0 – 6.0



Appendix E.6 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.

Appendix E.7 Conducting Other Performance Tests

For design traffic class TC80, selection of the design mix should be enhanced by undertaking a wheel tracking test (AASHTO T 324) to verify the rut resistance of the mix and the four-point bending beam test (AASHTO T 321) to guard against fatigue cracking.

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