REPUBLIC OF KENYA

MINISTRY OF ROADS AND TRANSPORT

RDM 5.2

Road Design Manual

Volume 5: Pavement Maintenance, Rehabilitation and Overlay Design

Part 2: Pavement Maintenance, Rehabilitation & Overlay Designs

2025

Chief Engineer (Materials)
Materials Testing & Research Division
State Department for Roads
Ministry of Roads and Transport
Machakos Road, Industrial Area
P.O. Box 11873 – 00400
NAIROBI

Principal Secretary
State Department for Roads
Ministry of Roads and Transport
Works Building
Ngong Road
P.O. Box 30260 – 00100
NAIROBI

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. This second generation of the 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 the provision of road infrastructure that is safe, secure, and efficient.

Under the Kenya Vision 2030 long-term plan, infrastructure expansion and modernisation are some of the foundations for the realisation of economic, social and political transformation of Kenya 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 of 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 Plans (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 under MTP I, II and III, respectively, totalling 17,450 km. This was a massive infrastructure development program intended to double the paved road network in 10 years compared to 8,600 km developed from independence in 1963 to 2008.

Implementation of MTP I to III resulted in the construction of 14,000 km of paved roads, which extended the paved road coverage to Arid and Semi-Arid regions, that had been previously neglected. However, some key milestones of the Vision 2030 goals have not been realized. This has been due to internal and external challenges. External challenges included: climate change – prolonged droughts; the emergence of 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 costs and delays in payments of land acquisition; lack of harmonisation of cross-border transport regulation and operational procedures; rapid urbanisation; increased traffic volume with 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 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 to the aspirations of the Kenya Vision 2030 and the Kenya Kwanza Government's 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 through 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 6,000 km of new roads, maintaining rural and urban roads, rail, air and seaport facilities and services; expand communication and broadcasting systems; and promote 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

The plan 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 with the Government's strategy will provide guidance 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 Government 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.

On behalf of the Government of Kenya, I would wish to thank the European Union for financing the development of the first drafts of the manuals in 2009 and the African Development Bank for the financial support in the review and updating of the manuals. I would also like to thank the members of the National Steering Committee and the Technical Task Force for their input. The Technical Administrators, and the Kenya National Highways Authority (KeNHA) for the procurement and able administration of the consultancy Contract. I also thank the Consultant, TRL Limited for their role in providing technical expertise that was essential for the success of the manuals updating exercise. I also wish to express my deepest appreciation to our stakeholders, and all those who have contributed to this process and the staff of the Ministry for their continued input.

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

Preface

Infrastructure development is key to successful economic delivery, therefore all existing infrastructure must be maintained or rehabilitated to enable it to continue servicing the end users safely and efficiently. A road is designed to provide good service for many years and therefore, good maintenance, planning and long-term management are essential. These activities rely on skilled human resources, collecting and maintaining records of historical and performance information and data.

This manual contains methods and procedures that Kenya has adopted to standardise pavement condition surveys for the management of road networks, as well as pavement maintenance, rehabilitation and overlay designs on highways, urban and rural roads .

The manual adopts and encourages context-sensitive and comprehensive conditions assessments, a concept that seeks to inform maintenance and rehabilitation design strategies that combine engineering best practices in harmony with the natural and built environment whilst meeting the required constraints and parameters surrounding each project.

Users of the manual are expected to follow the standards set here-in and seek approval of the Ministry should any departures be warranted.

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 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

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

Amendment No.	Description	Amendment Effective Date	Amendment Approved by

Acknowledgements

This Manual was prepared by the Ministry of Roads and Transport, State Department for Roads, with the kind assistance of the African Development Bank.

A National Steering Committee was set up and chaired by the Permanent Secretary, Ministry of Roads and Transport, with the following membership: Principal Secretary for Devolution, Office of the Deputy President; Chief Executive Officer, Inter-Governmental Relations Technical Committee; Chief Executive Officer, Council of Governors; Managing Director and Council Secretary, Kenya Bureau of Standards; Director, National Transport and Safety Authority; Director General, Kenya Roads Board; Director General, Kenya Wildlife Services; Chief Executive Officer, Engineers Board of Kenya; Director General, Kenya Rural Roads Authority; Director General, Kenya Urban Roads Authority; President, Institution of Engineers Kenya; Director Policy, Strategy and Compliance; Kenya National Highways Authority; Chief Engineer, Roads Division, State Department for Roads; Chief Engineer, Materials Testing and Research Division, State Department for Roads.

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.).

The Consultant for the review and updating of the manuals and specification for road and Bridge Construction was TRL Limited (UK), in partnership with Consulting Engineers Group, India and Norken International Limited, Kenya. The Consultant's team composed of the Team Leader, Charles T. Bopoto (Eng.), Author, Kenneth Mukura (Eng.), Assistant Author, Tinashe Nyabunze (Eng), and Project Manager, Warsame Mohamed.

Technical Administration was provided by James Kung'u (Eng.), Joachim Mbarua (Eng.) and Stephen K. Kogi (Eng.), assisted by the project secretariat Esther E.O. Amimo (Eng.), Monicah Wangare (Eng.) and Catherine K. Ndinda (Eng.)

Project coordination was provided by the KeNHA team led by Kungu Ndungu (Eng.) and J. N. Gatitu (Eng.), supported by Victoria Okumu (Dr.), (Eng.), Isaiah Onsongo (Eng.), Clarence Karot (Eng.), Howard Ashihundu (Eng.), Naomi Njoki Nthiga (Eng.), Shiphrah Mibey (Eng.), Mateelong Moses (Eng.) and Rose Rahabwanjohi (Eng.).

Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
CrI	Cracking Index
CSO	Crack, Seat and Overlay
d/D	Deflections
DBM	Dense Bituminous Macadam
DCP	Dynamic Cone Penetrometer
DEM	Dense Emulsion Macadam
DI	Deformation Index
ESA	Equivalent Standard Axles
ETB	Emulsion Treated Base
Ɛt	Tensile Strain
Ɛv	Vertical Strain
FWD	Falling Weight Deflectometer
GDP	Gross Domestic Product
HBM	Hydraulically Bound Materials
HMA	Hot Mixed Asphalt
HVR	High Volume Roads
HWD	Heavy Weight Deflectometer
LB	Labour Based
LVR	Low Volume Road
LWD	Light Weight Deflectometer
M CESA	Million Cumulative Standard Axles
MESA	Million Equivalent Standard Axle
NMT	Non-motorised Traffic
PPI	Pothole/Patching Index
RDM	Road Design Manual
SG	Subgrade
Std	Standard
TC	Traffic Loading Class
TRL	Transport Research Laboratory
VEF	Vehicle Equivalence Factor
σt	Tensile Stress
σv	Vertical Stress

Glossary of Terms

Aggregate	Hard mineral elements of construction material mixtures, for example: sand, gravel (crushed or uncrushed) or crushed rock.
Asphalt	Is commonly used as short hand for asphaltic concrete which is any design of high-quality bitumen / aggregate mixture.
Asphalt Concrete (AC)	A mixture to predetermined proportions of aggregate, filler and bituminous binder material plant mixed and usually placed by means of a paving machine. This term is used for all mixtures of this type including AC and Dense Bitumen Macadam (DBM).
Asphalt Surfacing	The layer or layers of asphalt concrete constructed on top of the road base.
Base Course	This is the main component of the pavement contributing to the spreading of the traffic loads. In many cases, it will consist of crushed stone or gravel, or of good quality gravelly soils or decomposed rock. Bituminous base courses may also be used (for higher classes of traffic). Materials stabilised with cement or lime may also be contemplated.
Binder Course	The lower course of an asphalt surfacing laid in more than one course.
Bitumen	The most common form of bitumen is the residue from the refining of crude oil after the more volatile material has been distilled off. It is essentially a very viscous liquid comprising many long-chain organic molecules. For use in roads, it is practically solid at ambient temperatures but can be heated sufficiently to be poured and sprayed. Some natural bitumen can be found worldwide that are not distilled from crude oil but the amounts are very small in comparison.
Bound Pavement Materials	Pavement materials held together by an adhesive bond between the materials and another binding material such as bitumen, lime or cement.
Capping Layer	(Selected or improved subgrade). The top of the embankment or bottom of excavation prior to construction of the pavement structure. Where very weak soils and/or expansive soils (such as black cotton soils) are encountered, a capping layer is sometimes necessary. This consists of better-quality subgrade material imported from elsewhere or subgrade material improved by stabilisation (usually mechanical) and may also be considered as a lower-quality sub-base.
Carriageway	Portion of the roadway including the various physically contiguous traffic lanes and auxiliary lanes, serving one or both directions of traffic, and not including shoulders.
Channelisation	The use of pavement markings or islands to direct traffic through an intersection.
Chippings	Stones used for surface dressing (treatment).
Climate Mitigation	The prevention or reduction of the carbon footprint of systems or processes.
Climate Resilience	The ability of infrastructure to resist the impacts of climate change and adverse or extreme weather events including the possibility for quick restoration following disasters.
Cold Milling	Planning of parts or whole layers for replacement of recycling or levelling at ambient temperatures
Collector	A standard of road that is characterised by an approximately even distribution of access and mobility functions.
Collector Distributor	A road used at an interchange to remove weaving from the through lanes and to reduce the number of entrances to and exits from the through lanes.

Glossary of Terms *(continued)*

Design Period — The projected period of time that an initially constructed or rehabilitated pavement structure will perform before reaching a level of deterioration requiring more than routine or periodic maintenance.

Design Period of a Pavement — The period of time that an initially constructed or rehabilitated pavement structure will perform before reaching a level of deterioration requiring more than routine or periodic maintenance.

Design Year — The last year of the design life of the road or any other facility, often taken as twenty years although, for costly structures such as major bridges, a longer period is usually adopted.

Equivalent Standard Axles (ESA) — A measure of the potential damage to a pavement caused by a vehicle axle load expressed as the number of equivalent 80 kN single axle loads that would cause the same amount of damage. The ESA values of all the traffic are combined to determine the total design traffic for the design period.

Evolved pavements — A pavement where the pavement has undergone stage construction or has been strengthened or widened or reconstructed over a period of years.

Expressway — A multilane, divided highway with a minimum of two lanes for the exclusive use of traffic in each direction and full control of access without traffic interruption.

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

Flexible Pavements — Pavement with a bituminous surfacing and with a base layer with or without a hydrocarbon binder. It includes those pavements that have a bituminous (surface dressing or asphalt concrete) surface. The terms "flexible and rigid" are somewhat arbitrary and were primarily established to differentiate between asphalt and Portland cement concrete pavements.

Full reconstruction — Maintenance treatment that involves the replacement of all the pavement layers

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

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

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

Where: $P_{2.36}$ = percentage passing 2.36 mm sieve.
$P_{0.425}$ = percentage passing 0.425 mm sieve.
$P_{0.075}$ = percentage passing 0.075 mm sieve.

Heavy Goods Vehicles (HGV) — Vehicles having an unloaded weight of 3500 kg or more.

Hot Mix Asphalt (HMA) — This is a generic name for all high-quality mixtures of aggregates and bitumen that use the grades of bitumen that must be heated in order to flow sufficiently to coat the aggregates. It includes Asphaltic Concrete, Dense Bitumen Macadam and Hot Rolled Asphalt.

Hydraulic Stabilisation — Stabilising granular materials using (hydraulic) binders cement or lime.

Inlays — Milling and replacing existing layer(s) and replacing them with new or recycled material to the same level as the original layer(s)

Overlay	One or more courses of asphalt construction on an existing pavement. The overlay often includes a levelling course, to correct the contour of the old pavement, followed by a uniform course or courses to provide needed thickness.
Partial reconstruction	Replacement of all the bound layers.
Pavement Layers	The layers of different materials form the pavement structure.
Pavement maintenance	Regular inspection of pavement for damages and repairs carried out to prevent extensive future work or accidents
Pavement Rehabilitation	Pavement surface and structural condition evaluation, and the design and execution of works to restore the pavement to the desired condition and reset the deterioration process.
Reconstruction	The process by which a new pavement is constructed, utilising mostly new materials, to replace an existing pavement.
Recycling	The reuse, usually after some processing, of a material that has already served its first intended purpose.
Reprocessing	Scarification or milling of existing pavement layers followed by reworking and compaction.
Rigid pavements	Pavement whose primary mode of failure is cracking and fracture or faulting and formed of concrete or thick asphalt or block paving and characterised by low deflections.
Road base	A layer of material of defined thickness and width, constructed on top of the sub-base, or in the absence thereof the subgrade. A road base may extend to outside the carriageway.
Severe pavements	Pavement under severe conditions such as climbing lanes, and pavements in flood-prone or hot climates.
Side Slope	Area between the outer edge of the shoulder or hinge point and the ditch bottom.
Surface treatment	Any treatment required to rectify surface defects on the wearing course or surfacing of bituminous pavements or shallow defects on rigid pavements or repairs on the gravel wearing course.
Unbound Pavement Materials	Naturally occurring or processed granular material which is not held together by the addition of a binder such as cement, lime or bitumen.
Unpaved roads	Road with a granular surfacing.
Vehicle Equivalency Factors	Used to convert traffic volumes into cumulative equivalent standard axle loads per vehicle category.
Walkway	The portion of the cross-section that is reserved for the use of pedestrians.
Wearing Course	The top course of an asphalt surfacing or, for gravel roads, the uppermost layer of construction of the roadway made of specified materials.

Contents

List of Figures

List of Tables

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 incorporates best practices, climate change considerations, and recent technologies to enable the provision of road infrastructure that is safe, secure, and efficient.

The road manuals system is composed of:

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	

This table continues onto the next page...

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 and 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 and Social Inclusion Monitoring	

This Volume 5, Part 2 – 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 and Culvert Design	RDM 4.2
		Part 3 – Retaining Structures Design	RDM 4.3
		Part 4 – Reinforced Fill Structures Design	RDM 4.4
		Part 5 – Bridges and 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 Objectives of this Part

The objective of RDM 5.2 is to guide the design engineers in the evaluation of existing road pavements by qualifying their functionality and quantifying their performance, analysing their condition, evaluating their capacity and determining mitigating interventions. The content provided is designed to enable users to quality and characterise the pavements and their materials to develop parameters required for maintenance and rehabilitation design.

Additionally, it is also aimed at providing comprehensive technical content to enable the designers to carry out critical analysis and to prepare cost-effective pavement maintenance, rehabilitation and overlay design. This part has been specifically prepared to allow for innovation and creativity as well as the adaption of technological advancements in line with best practices while operationalising the standards provided.

1.3 Scope of This Part

RDM 5.2 covers procedures of rational management, maintenance, rehabilitation and overlay design of both sealed and unpaved roads. It sets out a framework within which established assessment and design techniques and engineering judgement are to be applied.

This Part is applicable at the stage of the design process where the review of the network condition and safety data has shown that a significant length of a road has problems and defects that may require substantial remedial work. This shall then warrant detailed traffic, functional and structural condition surveys (RDM 5.1), and pavement evaluation, maintenance and rehabilitation design, which are covered in detail in this Part.

The content covered under this Part includes:

1. Consideration of information from pavement planning and management. This involves the review of historical information including as-built data, traffic volumes and traffic loading as well as performance data.
2. Traffic assessment – the determination of current and future traffic loading to be carried by pavement which is used for maintenance and rehabilitation design.
3. Pavement evaluation – the existing pavement is evaluated for its functional and structural condition as well as the quality of materials making up the pavement layers. This information is used to determine the adequacy of the pavement to carry future traffic.
4. Materials for overlay design – the materials to be used for the overlays are considered in detail and the information is used in the design of overlay thicknesses.
5. Overlay design methods – this Part covers 4 key design methods of overlay design of flexible pavements including the Catalogue Method, the Mechanistic Empirical Method, the AASHTO Design Method and the Performance Design Method. For overlay of rigid pavements, 4 design methods are provided including the Empirical Concrete Overlay Design Method, Modified Empirical Concrete Overlay Design Method, Structural Deficit Design Method (based on UK/ AASHTO design principles) and Mechanistic-Empirical Design Method (based on the India/ AASHTO Method).
6. Other rehabilitation design methods – these are methods other than overlays such as recycling, inlays, reseals and fog sprays.
7. Other key considerations – this Part provides other key aspects of maintenance and rehabilitation for the design engineer to consider such as climate resilience, roads in urban settings and unpaved roads.
8. Maintenance – Pavement maintenance is covered for both flexible and rigid pavements. Where overlays are not required maintenance activities must be carried out to treat the defects. The identification and assessment of defects are covered in RDM 5.1 and this Part covers their treatment.

RDM 5.2 does not cover routine pavement maintenance or minor repairs. It also does not cover sections under contractual warranties.

1.4 Organisation of this Part

This manual is made up of 15 Chapters laid out sequentially in the order in which the maintenance and rehabilitation work is carried out.

- Chapter 2: General Requirements for Pavement Maintenance and Rehabilitation
- Chapter 3: Planning for Pavement Maintenance and Rehabilitation Design
- Chapter 4: Assessment of Historical Information
- Chapter 5: Traffic Loading Analysis
- Chapter 6: Pavement Evaluation
- Chapter 7: Criteria for Maintenance and Rehabilitation
- Chapter 8: Techniques and Materials for Road Strengthening
- Chapter 9: Structural Design of Overlays for Flexible Pavements
- Chapter 10: Structural Design of Overlays for Rigid and Semi-rigid Pavement
- Chapter 11: Other Rehabilitation and Construction Techniques
- Chapter 12: Design Considerations for Climate Resilience
- Chapter 13: Design Considerations for Urban Roads
- Chapter 14: Considerations for Unpaved Roads
- Chapter 15: Pavement Maintenance

Appendices are provided and include Catalogues for overlay thickness design for different overlay materials depending on the strength of the existing pavements and forecasted traffic loading and a sample of the pavement types based on the foundation design approach given in RDM 3.3. Also included in the Appendices is a complete worked example of pavement evaluation and the overlay design process and outputs to provide clarity to the design engineers.

2 General Requirements for Pavement Maintenance and Rehabilitation

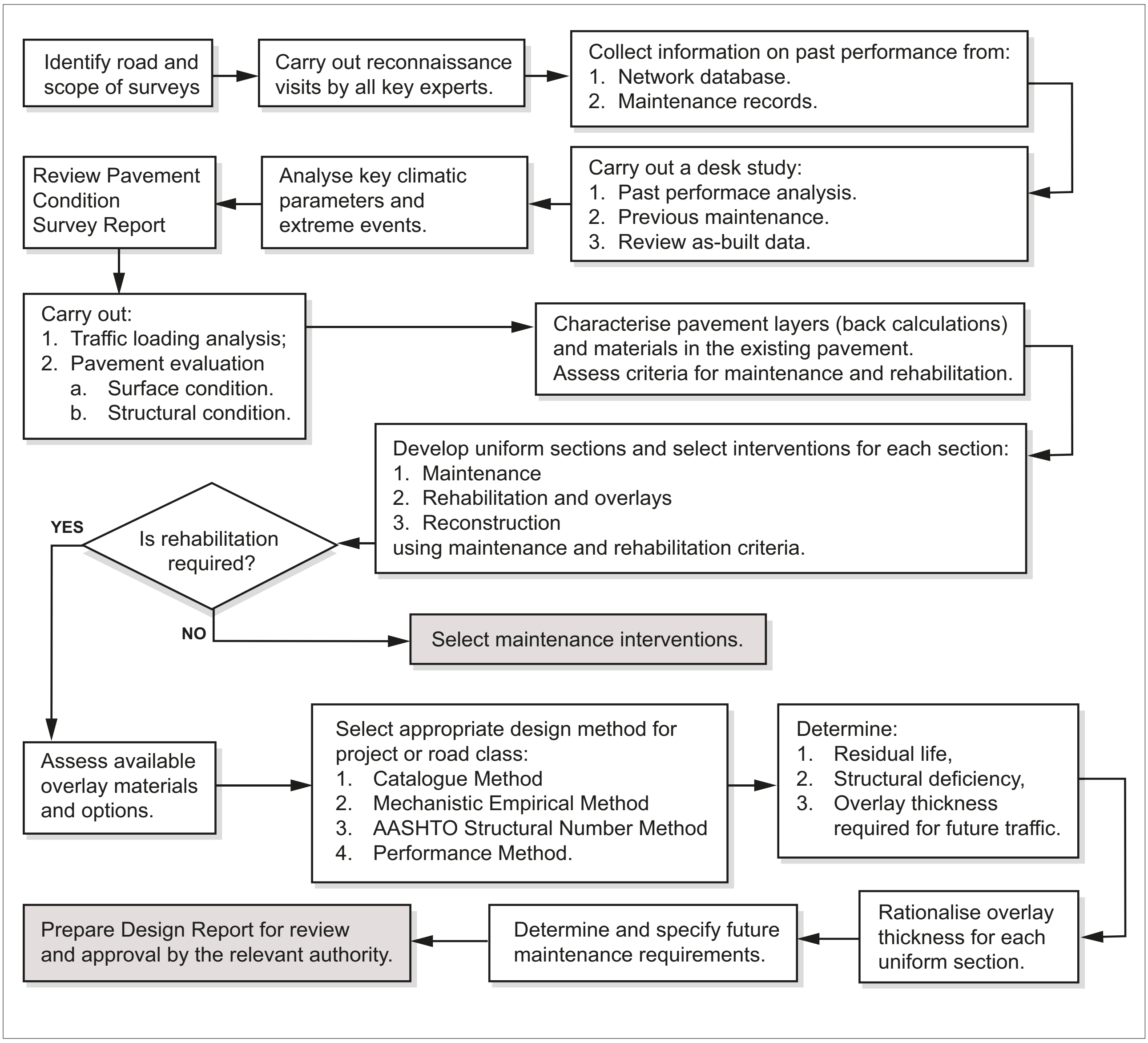
2.1. General

This Chapter covers key requirements for pavement maintenance and rehabilitation design. These requirements are necessary to ensure a robust and economical design and they are premised on a systematic design process given in the flow chart in Figure 2.1.

2.2 Design Process

Figure 2.1 provides a flow chart on key stages and elements of the design process, which should be followed for the solution options to be robust, economical, and sustainable while ensuring accountability.

Figure 2.1 General Requirements for Conducting Pavement Conditions Surveys



2.3 Scoping Maintenance and Rehabilitation Design Projects

The scope of the project should be properly defined and documented for the following reasons:

1. To inform pavement condition surveys on the types and quantum of data to be collected.
2. To evaluate the risks associated with the project considering the anticipated value of the works.

The project scope shall include the following key aspects:

1. Review of the network data on past performance to fully understand the reasons why the section was chosen.
2. Review of pavement condition surveys (RDM 5.1).
3. Planning for the designs – selection of the design options to be considered, which are appropriate for the scope of the project.
4. Pavement evaluation – assessing both the surface, structural conditions and residual life based on future traffic loading.
5. Characterisation of the pavement layers and materials.
6. Assessing various materials and options for the remedial works.
7. Carrying out the designs using guiding principles given in Chapter 9 and Chapter 10.
8. Evaluating maintenance and rehabilitation techniques that are appropriate for the size and class of the road as well as prevailing conditions, considering climate resilience and mitigation as well as the location of the road (urban or rural) and class of road unpaved or paved and high or low volume road.

2.4 Historical Data

2.4.1 Original Design and As-Built Data

The designer shall collect and review:

1. Original Design:
   a. Design reports and design review reports.
   b. Drawings.
   c. Specifications.
2. As-built data – including:
   a. Pavement structure, dimensions, deviations from designs and standards, etc.
   b. Construction – densities achieved, quality of materials used, site instructions, approvals, etc.
   c. Other – crossfalls, drainage systems/profiles, initial traffic, design traffic, design life, etc.

2.4.2 Pavement Performance Data

The designer shall collect and evaluate historical data on how the pavement performed and the prevailing conditions of traffic and the environment. This shall include:

1. Time to crack initiation.
2. Nature of defects and their progression over time.
3. Record of any premature failures.
4. Record of damage to the pavement.
5. Trends of deflection measurements.
6. Condition at the end of defects liability period.
7. Drainage condition.
8. Rate of deterioration of unpaved roads (rate of gravel loss and roughness progression).

2.4.3 Sources of Historical Data

Historical data shall be obtained from the following:

1. Road Agencies including KeNHA, KURA, and KeRRA.
2. The Kenya Roads Board (KRB).
3. Material Testing and Research Directorate (MTRD).
4. Kenya Bureau of Standards (KEBS).
5. Central Database or repository of Road Asset Management Systems.
6. Any research carried out by universities.

The names of these sources may change, and historical changes should also be considered.

2.5 Pavement Evaluation Techniques and Software

The steps for pavement evaluation should include the following:

1. Analysis and interpretation of the surface condition of the pavement in terms of the extent and severity as well as distribution of defects.
2. Development of uniform sections based on the pavement inventory, traffic surveys and functional condition surveys.
3. Analysis of the structural condition of the pavement in terms of the structure and characteristics of the pavement, pavement layers, pavement materials, moisture condition, etc.
4. Analysis of influencing factors for the deterioration such as traffic (both present and future traffic), the environment and climate change impact (climate resilience).
5. Software used for back calculation to determine the strength characteristics of the layers and overall pavement traffic load carrying capacity or residual life.

2.6 Applicable Design Methods

The main design methods covered in this manual, which are detailed in Chapter 9 and Chapter 10 include:

For flexible pavements:

1. Catalogue Method
2. Mechanistic-Empirical Method
3. AASHTO Structural Number Method
4. Performance Method

For rigid pavements:

1. Section 10.6: UTRCP (Ultra-Thin Reinforced Concrete Pavement) where a fixed 50mm thick. This is effectively a non-structural overlay.
2. Section 10.7: Standard UK Method
3. Section 10.8: UK Method (modified for Kenya)
4. Section 10.9: Structural Deficit Method (based on UK/AASHTO design principles)
5. Section 10.10: Mechanistic-Empirical Design Method (based on the India/AASHTO Method)

The criteria to be considered for selection of the appropriate design method are:

1. Availability of historical data especially for the Performance Method (also known as the Deflection Reduction Method)
2. Availability of equipment and software (especially for the Mechanistic-Empirical Method).
3. Class of road. – For medium and heavy-traffic roads, the Mechanistic-Empirical Method is recommended.

The design processes, especially for the mechanistic method, are iterative and cumbersome. Computer-aided design is required particularly on medium to large projects. The designer should select suitable software.

3 Planning for Pavement Maintenance and Rehabilitation Design

3.1 General

This section covers the project planning required for a maintenance and rehabilitation design process. The planning involves 4 key stages:

1. Project appraisal – to determine the economic viability of the project. Refer to Project Appraisal Documents/Manual for details.
2. Environmental and social impact assessment (ESIA) – this is meant to minimise project impacts on the environment (Refer to the Environmental and Social Impact Assessment Manual).
3. Traffic Assessment – information on traffic volumes by categories is important for rehabilitation design (RDM 1.2 and RDM 5.1).
4. Traffic loading – this emanates from the axle load surveys. (RDM 1.2 and RDM 5.1).

3.2 Planning Period

It is necessary to anticipate the need for major inputs to the network far enough in advance to allow for design activities, finance allocation and the assembly of resources to be completed on schedule before a project starts. Minimum planning periods for interventions are given in Table 3.1.

Planning periods could be significantly reduced through a functional pavement management system (PMS), where pavement performance data are collected regularly.

Table 3.1 Planning Periods of Interventions With and Without PMS.

Item no.	Intervention	Planning period without PMS (Years)	Planning period with data from PMS (Years)
1	Periodic maintenance	2	1
2	Extensive rehabilitation and strengthening	4	2

Usually, uncertainties exist due to inaccurate of pavement deterioration predictions because of unexpected increases in traffic and adverse weather conditions.

Accordingly, a 'stop-gap' maintenance input should be authorised to enable remedial work to be carried out at short notice.

Sections of the network that are due for maintenance within the first year of the planning period should be identified and dealt with urgently. It may be justified to implement short-term measures to arrest deterioration. This is a staged-construction approach to suit budgets.

3.3 Priorities for Consideration

Ideally, the whole network should be assessed annually for its functional and structural performance, though such extensive coverage may not always be possible. Detailed assessments should be carried out on key sections of the road network, and the remainder in less detail and frequently. The intensity of the assessments should depend on the functionality of the section, traffic volumes and the complexity of the pavement structure. Special consideration should always be given to sections in poor condition regardless of other factors.

3.4 Project Appraisal

Project appraisal generally comes before rehabilitation design and involves the assessment of the viability of the project mostly financially. Project costs are an important element of this.

3.4.1 Total Transport Cost Components

The principal economic criterion is to optimise investment based on 'total cost' which comprises capital and recurrent costs.

1. **Capital costs:** these are usually well-defined and isolated in nature such as overlay or reconstruction operations.
2. **Recurrent costs:** these are continually incurred costs during the operation period. Some are less well-defined, for instance, accidents and environmental impacts cannot readily be assigned economic values.

There are invariably recurrent costs throughout the usage of the road. In some analyses, however, the total cost may not always include a capital cost. This is the case in the 'do nothing' option, which must be evaluated as a base for comparison with any prospective investment under consideration.

3.4.2 Total Cost Components

The major components contributing to the total transport cost are:

1. **Construction Costs:** if some improvements are considered.
2. **Vehicle Operating Costs (VoCs):** dependent on route length, volume and type of vehicles utilising the road and their operational unit costs.
3. Road Maintenance Costs: expected recurrent costs for routine and periodic maintenance inputs.
4. **Further Investment Costs**: expected cost at the end of the current design period for the road to handle the forecasted traffic demand.

The net present value of these costs is calculated by discounting them to a chosen base year. This is usually considered to be the first year of the appraisal.

3.4.3 Vehicle Operating Costs Considerations

The implementation of the management system, should reduce the unit running costs of vehicles. In most cases, vehicle operating costs (VoCs) per unit distance in terms of fuel consumption, electricity consumption (for electric vehicles), tyre replacement, spare parts and maintenance labour are dependent on the condition of the running surface and the average vehicle operating speed. The latter is directly influenced by road characteristics such as the topography, geometry and carriageway width.

3.4.4 Cost of Accidents

Similarly, the occurrence and severity of accidents are affected by differences in the alignment, carriageway width and surface condition. Prediction of accident type and frequency can be obtained for the Kenya Roads Board (KRB) road conditions and only the effect of very substantial changes may be forecast. Moreover, whilst significant resource costs are incurred through accidents, other important consequences, for instance, personal injury or loss of life, cannot yet be given financial value.

3.4.5 Associated Investments

It may happen that the need for one investment (for instance strengthening), will trigger the justification for other associated investments such as pavement widening, realignment, etc. Effective cost saving may result from co-ordinating the investments and/or interchanging their sequence of execution. It is important to hold an overall view and to appreciate the significant implications of one investment on other possible associated investments. Table 3.2 presents the likely linkages of investments.

The main consequences expected from different actions for each investment are summarised in Table 3.3.

Table 3.2 Possible Co-Ordination of Associated Investments for Net Benefit

Investment type under consideration	Associated investments, which offer cost savings if co-ordinated
Rehabilitation and reconstruction	Realignment (to improve geometric standards)
Drainage/shoulder improvement or repair	Pavement widening
Surface treatment (e.g. resealing or recarpeting)	Strengthening
	Pavement repairs
Strengthening	Shoulder improvement -Pavement widening
Rehabilitation	Strengthening -Pavement widening
Reconstruction	Pavement widening -Realignment -Shoulder improvement
Pavement widening	Surface treatment

Table 3.3 Expected Outcomes of Various Investments

Action	Effect	Outcomes
Drainage repair or improvement	Reduces pavement deterioration. Reduces the probability of washout, etc.	Reduces future vehicle running costs. Increases route reliability. Reduces maintenance costs.
Shoulder repair or improvement	Increases the effective width of the pavement. Reduces pavement deterioration	Reduces accidents. Reduces vehicle running costs. Reduces maintenance costs
Surface treatment	Improves running surface condition. Reduces pavement deterioration.	Reduces vehicle running costs. Reduces maintenance costs
Pavement strengthening	Renews running surface condition. Reduces pavement deterioration. Increases pavement bearing capacity.	Reduces vehicle running costs. Reduces routine maintenance costs. Increases the life of the pavement.
Pavement rehabilitation	Renews running surface condition. Reduces pavement deterioration. Increases pavement bearing capacity.	Reduces vehicle running costs. Reduces routine maintenance costs.
Pavement widening	Increase the effective width of the pavement. Improve horizontal alignment.	Reduces accidents and increase route capacity.
Reconstruction	Any or all of the above, improve vertical alignment.	Any or all of the above.
Realignment	Any or all of the above, changes route length. Improves horizontal and vertical alignment.	Any or all of the above. Changes vehicle running and unit costs. Changes maintenance costs.

3.4.6 Future Investments

The further investment components should be estimated for the most probable continued requirement of the road, aiming at restoring the road to the same final state for each of the alternative strategies considered in the appraisal.

The cost of achieving this will vary for the different strategies. Thus, in one strategy, the road might be in a suitable condition to be overlaid to handle the expected traffic, whereas in another strategy the road might require complete reconstruction for similar traffic.

The purpose of further investment cost elements is to reflect any differences in the residual value of the road because of different strategies. Thus, although speculative long-range estimates are involved, the differences are a sensible reflection of the residual value in one strategy compared with another. In any case, the residual value should be discounted over the full appraisal period.

For economic appraisals, all costs may be calculated excluding the taxes and transfer payments. The Chief Engineer (Roads) can advise on current factors applied to construction, vehicle operating costs, road maintenance etc. to estimate economic costs.

4 Assessment of Historical Data

4.1 General

Ideally, there should be a continual reappraisal of the use of each section of the network and how effectively it performs its functions. The situation on heavily used sections may warrant more frequent or intensified reappraisal. Table 4.1 shows the recommended criteria to be considered during the assessment.

Table 4.1 Key Information Required for Maintenance and Rehabilitation From Pavement Network Assessment

Item No.	Investment type under consideration	Associated investments, which offer cost savings if co-ordinated	
1	**Strategic functions** (i.e., classification)	**a.**	Network improvement needs.
		b.	Traffic flow patterns
2	**Geometric standards**	**a.**	Accident/safety surveys
		b.	Level of service (LoS)
3	**Maintenance levels and costs**	**a.**	Inventory updating
		b.	Performance specifications
4	**Economic viability**	**a.**	Structural performance
		b.	Axle load distribution
5	**Rehabilitation**	**a.**	Prioritisation
		b.	Resource allocation

Considerations such as route classification will rarely change, whereas traffic flow characteristics will be subject to steady and predictable changes unless special influences occur (e.g. revision of traffic legislation or substantial change in fuel prices, etc.). The pavement condition and subsequently the structural performance are generally less predictable, particularly, on roads with heavy traffic, regular monitoring is necessary so that the onset of pronounced deterioration can be anticipated.

Essentially, the management task is to determine the proper nature, strategies and timing of investments in the road network.

4.2 Assessment of Historical Data and Information

4.2.1 General Requirements

Road networks are high-value national assets, which should be managed to ensure sustainability, durability, reliability and safety. To achieve this, regular data and information collection should be carried out and budgeted. Historical data that should be collected for pavement maintenance and rehabilitation and the approach to acquiring the data are provided in RDM 5.1. These data should be obtained from the Chief Engineer Materials and the relevant road authorities.

Data from regular surveys are used to determine performance trends and develop or calibrate deterioration models. At the pavement maintenance and rehabilitation design stage, historical data provides the following key parameters:

1. **Updated inventory** – components of infrastructure to be considered in rehabilitation design.
2. **Traffic growth rates** – for traffic loading projections.
3. **Traffic loading parameters** – for distribution of traffic loads, and development of traffic-loading factors.
4. **Residual life of roads** – the consumption of the life of the pavement through the determination of the cumulation of passages of axle loads. The result is used to determine residual life.
5. **Calibration of fatigue relationships for pavements** – Predicted performance is compared with actual performance of pavements.

6. **Determination of preventive maintenance** – where rehabilitation is not required.
7. **Impacts of environmental factors** – deterioration or damage caused by environmental factors.
8. **Historical performance data** – indicates the progression of defects and general pavement behaviour.
9. **Life cycle costs** – a key element of project appraisal.
10. **Defects analysis and pavement condition** – these are used as triggers for various interventions when standard thresholds are exceeded.

A typical example is the Performance or Deflection Reduction Overlay Design Method given in Section 9.8, which involves the use of historical deflections and rut depth data for the calculation of overlay thicknesses.

4.3 Maintenance Rehabilitation and Upgrading Needs of Pavements

The purpose of a pavement management system is to determine from routine monitoring measurements, the types of maintenance operations or rehabilitation required for each section of the network or road link.

In most cases, no action may be required. In some sections, the assessment of the pavement condition will indicate the needs given in Table 4.2, which should be considered by the engineer during pavement maintenance and rehabilitation design.

Table 4.2 Maintenance, Rehabilitation and Upgrading Needs for Pavements.

Item No.	Level of Maintenance Required	Interventions	
1	Routine maintenance	a.	Crack sealing and pothole patching when they occur and routinely before the rainy season.
		b.	Edge break repairs when they occur.
2	Periodic maintenance	a.	Fog sprays at about 4-year intervals
		b.	Reseals once the surfacing begins to show cracks, ravelling, and low skid resistance.
	Surface treatments	c.	Recarpeting
		d.	Application of maintenance slurry seals
		e.	Application ultrathin concrete surfacing
3	Rehabilitation	a.	Strengthening through overlays
		b.	Retrofitting climate resilience
4	Recycling	a.	Mainly for AC – mill, process and mix with new constituents of binder and aggregate or recycling agent to soften the binder and relay as binder course, base or subbase.
		b.	For rigid pavements – mill or crack and sit as base and overlay with new concrete or asphalt.
5	Inlays	a.	For AC – Mill and replace with new asphalt of equivalent or higher strength.
		b.	For rigid pavements – mill and replace with new concrete of equivalent or higher strength
6	Reclaimed asphalt	For AC/binder course – mill, reprocess and use granulated asphalt as base or subbase or fill or even landfill or for preparation of recycled asphalt.	
7	Reprocessing	Mainly for granular layers – excavate or mill, remix and recompact.	
8	Reconstruction	This involves reprocessing and replacement of some pavement layers including the subgrade.	
9	Upgrading	a.	Improvement of alignment
		b.	Road widening.

Surface treatment includes resealing/recarpeting, and applying micro-surfacing, which waterproofs and minimises loss of volatiles and oxidation of underlying pavement layers and thus decelerates the deterioration, but without improving the structural strength of the pavement.

Surface condition surveys, when considered together with deflection measurements, will show some structural weakness developing in the pavement and that a substantial action other than simple surface treatment will be appropriate. Consequently, depending on the nature and extent of the pavement distress, it may be concluded whether the particular section requires rehabilitation, strengthening or reconstruction.

5 Traffic Loading Analysis

5.1 Traffic Growth Rates

Traffic growth rates are derived from several sources:

1. Historical traffic counts.
2. GDP growth (percentage).
3. Vehicle registration.
4. Fuel consumption.

The GDP growth rate averaged over 3 to 5 years is used as a good approximation in the absence of other data.

For more details on traffic counts, categorisation, and projecting traffic volumes and traffic loading refer to RDM 1.2, RDM 5.1, RDM 3.3 and RDM 3.4.

A summary of the calculation of cumulative traffic loading or design traffic is given below. A more detailed procedure for the computation of vehicle equivalency factors and the design traffic is presented in RDM 3.3 and RDM 3.4.

5.2 Calculation of Equivalent Standard Axles Load Factors (ESALFs)

Calculate the equivalent standard axles ($ESALF$) using Lidle's formula (Equation 5.1):

$$ESALF = \left(\frac{P}{8160}\right)^{n}$$

Equation 5.1

Generally, an exponent of 4.5 (n = 4.5) shall be used.

Determination of VEFs shall be done by measuring each axle and summing up the $ESALF$s (obtained using Equation 5.2 and Equation 5.3).

Single axle with dual wheels on both sides.

$$ESALF = \left(\frac{P}{8160}\right)^{4.5}$$

Equation 5.2

VEF is the summation of ESAs for each axle i, ……., k

$$VEF = \sum_{i}^{k}(ESALF_i)$$

Equation 5.3

Where,

P = Axle load, kN.

n = Damage factor/power exponent.

k = Number of axles on the vehicle.

The standard load is in kN

1
2
3
4
5
Traffic Loading Analysis

5.3 Calculation of Cumulative Equivalent Standard Axles (CESA)

Traffic estimation/forecasting – traffic estimation is carried out in 4 ways.

1. Initial traffic – at the time of the traffic counts, accumulated over the design period. The analysis should include traffic growth.
2. Diverted traffic – traffic that would divert from adjoining routes of the road network due to new preferences for the route under design attracted by better riding quality and travel time savings.
3. Generated traffic – additional traffic generated by demand and increased socio-economic activities along the route and key connector.
4. Changes/revolution of modal transport systems

Use Equation 5.4, Equation 5.5 and Equation 5.6 to calculate cumulative traffic loading:

$$T_i = \frac{365 \text{ x } A_i \text{ x}\left[\left(\frac{r}{100}\right)^N - 1\right]}{\frac{r}{100}} \text{ x } VEF \text{ x } LDF$$

Equation 5.4

Traffic in the year of completion, A:

$$A_i = P_i\,(1 + r)^x$$

Equation 5.5

The total number of ESAs, T, in CESA:

$$T = \sum_i^k T_i$$

Equation 5.6

Where,

T_i = cumulative traffic loading in category (i), CESAs
r = traffic growth rate, %
N = design life, years
LDF = Lane distribution factor, %
VEF = Vehicle equivalent factor
A_i = Actual traffic in the year of completion
P_i = Last traffic count
x = Number of years between the last count and years of completion.
k = Number of classes of commercial vehicles

The lane distribution factor considers the distribution of the repetitions of the axle loads by:

1. Lane in a multilane system – which in turn may be influenced by traffic regulations e.g., where light vehicles use the inner lanes and heavier vehicles use the outer lanes. The loading and response of the pavement would be different in each case.
2. Within each lane – wandering of axles loads within each lane where:
   a. For narrow lanes – channelisation occurs.
   b. For wider lanes - wandering of load repetitions occurs thus reducing the rate of fatigue-related deterioration, see Section 9.4 and Section 9.6.

Guidance on lane distribution factors is given in Table 5.1. Where traffic loading measurements have been carried out for each lane, the measured traffic loading distribution should take precedence over these factors.

Table 5.1 Lane Distribution Factors for Different Classes of Roads

Item No.	Carriageway Description	Description of Lane Vehicle Distribution	Lane Distribution factor, LDF
1	**Single lane roads**	Use a total number of passes (100%) of commercial vehicles because traffic is channelised.	1
2	**2-lane single carriageway**	Use 50% of CESAs of commercial traffic because it is shared between the 2 lanes. If one lane is carrying more traffic than the other, use the lane with the higher figure.	0.5
3	**4-lane single carriageway**	Use 40% of the total commercial traffic CESAs in both directions.	0.4
4	**Dual 2-lane carriageway**	Use 75% of CESAs in each direction	0.75
5	**Dual 3-lane carriageway**	Use 60% of CESAs in each direction	0.6
6	**Dual 4-lane carriageway**	Use 45% of CESAs in each direction	0.45

5.4 Traffic Loading Classes

The traffic loading classes that should be considered in rehabilitation design are given in Table 5.2. The classes are used to determine pavement overlay thicknesses given in the catalogue method, and the material specifications for overlays and VEFs for each class.

Table 5.2 Traffic Classes for Consideration in Rehabilitation Design

Design Traffic Class	Cumulative Equivalent Standard Axles	Design Value (MCESA)	Traffic Load Category	DESA: 5% growth rate for 20-year Design	
				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+	**> 150 million**	Based on specific value		>12428	

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

6 Pavement Evaluation

6.1 General

This section covers the evaluation of existing pavements. Pavement evaluation is necessary to determine its adequacy to carry future traffic in the given environment and to decide on the maintenance or rehabilitation measures, that may be needed to restore it to a good condition. Pavement evaluation includes a comparison of both surface condition ratings and structural ratings with the required level of service and structural capacity to accommodate future traffic loading.

6.2 Surface Condition Evaluation

Surface condition ratings indicate how well the road is serving the travelling public. Surface condition surveys provide valuable and necessary information but are insufficient to judge the structural adequacy of the pavement.

The results of the pavement surface condition surveys are mainly used to:

1. Assess the effects on the road user.
2. Establish the probable causes of surface distress.
3. Determine the need and priorities for maintenance operations and surface rehabilitation.
4. Determine the need for structural evaluation.
5. Assess the pavement deterioration rate, so that the approximate time for planning future work or carrying out another condition survey can be predicted.

A surface condition assessment can be based on one, or a combination of the following:

1. Measurements of surface distress, showing locations and extent of each defect observed.
2. Measurements of surface roughness.
3. Subjective rating of the pavement riding quality and surface condition.

A pavement assessment should always be related to the history of the pavement. In particular, it is essential to:

1. To record the maintenance operations carried out, since they may have modified the surface condition by temporarily concealing defects.
2. To assess and consider the function of the road concerned and the traffic, that uses it.

6.2.1 Recording and Quantifying the Defects

In the first stage, the defects should be visually identified and their extent estimated so that the road can be divided into sections, which exhibit similar defects and to a similar extent.

In the second stage, and at regular intervals, all defects should be systematically recorded, located and quantified, as indicated in RDM 5.1.

For a road that has been recently resealed, obtain records of the condition and defects before the maintenance intervention.

6.2.2 Surface Roughness

There are several established techniques for measuring surface roughness including:

1. MERLIN machine.
2. Bump integrator.
3. Roughometer.
4. Approved smartphone app.
5. Laser-based pavement surface profilers.

The bump integrator and other devices aggregate the total vertical movement of a wheel relative to its mounting frame as the wheel is towed at a standard speed along the road. The World Bank app uses the accelerometer in smartphones to integrate displacements.

The surface roughness is the aggregated measurement of displacements per unit length of road travelled (m/km).

6.2.3 Present Serviceability Concept

This concept, which was developed in connection with the AASHTO Road Test, presents serviceability as the ability of a specific section of road to provide a smooth, safe and comfortable ride at that particular time. A present-day serviceability value may be obtained by subjectively rating the pavement through visual observations (present serviceability rating) or by quantitative measurement of surface characteristics (present serviceability index).

6.2.3.1 Present Serviceability Rating (PSR)

PSR involves using of a group of raters who ride the pavement section, observe its riding quality, assess its condition and record their impressions on a standard form. The procedure for determining the PSR is detailed in RDM 5.1.

Ratings vary from “1” (very poor) to “5” (very good). Low ratings indicate poor surface condition and suggest a more detailed examination of the pavement is required. The PSR may be used as a first step in evaluating the adequacy of a pavement.

6.2.3.2 Present Serviceability Index (PSI)

The Present Serviceability Index is an equation that, when equated with a panel’s serviceability rating, can be used, together with results of measured surface defects and roughness, to quantify a road section’s rideability.

The PSI is mainly dependent upon the roughness of the pavement surface and consequently, a simplified PSI may be determined using Equation 6.1:

$$PSI = 0.500 - a(IRI) - b(log\ IRI)$$

Equation 6.1

Where, 'IRI' is roughness and 'a' and 'b' are coefficients (see note).

Notes:

1. *The above coefficients depend on the country, types of pavements analysed and the equipment used for measuring longitudinal profile variations or roughness.*
2. *A single PSI value is not itself a measure of absolute pavement performance but it is representative of the trend of serviceability that gives indications about the performance of the pavement.*

6.3 Structural Evaluation

The road pavement consists of layers including the subgrade or road foundation and pavement layers, comprising the subbase(s) and base(s), the binder course and the surfacing or wearing course. The configuration may differ depending on the original design, which may have been influenced by available materials, and the strength required to sustain the loading and the environmental factors.

The structure of pavements is illustrated in Figure 6.1, Figure 6.2 and Figure 6.3. These are generic and generally represent pavement structures of the existing road network built before the advent of the foundation design approach given in RDM 3.3, 3.4 and 3.5. Pavements, that are designed based on the foundation design approach will be structured as illustrated in Figure 6.7 (for flexible pavements).

Figure 6.1 Structure of Flexible Pavements

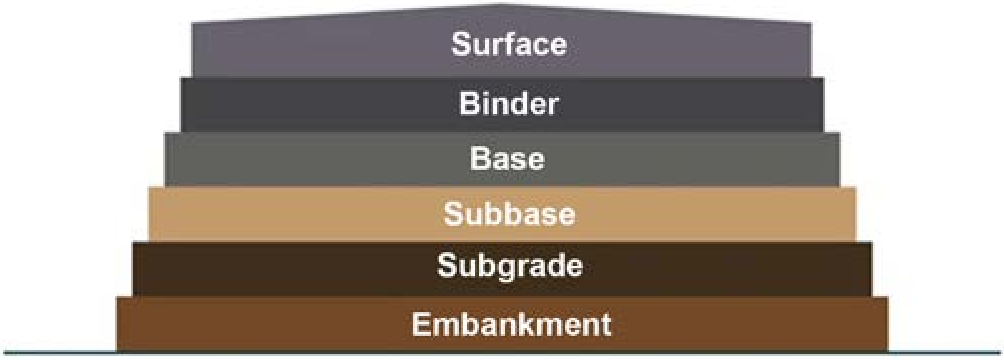


Figure 6.2 Structure of Rigid Pavements

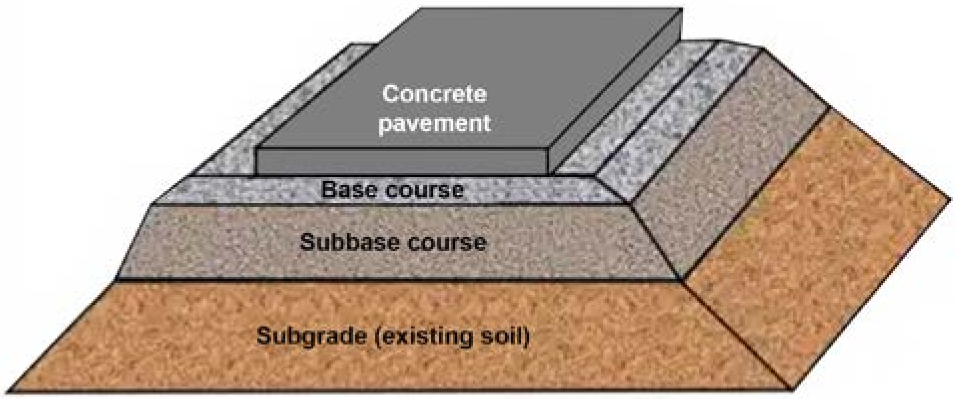
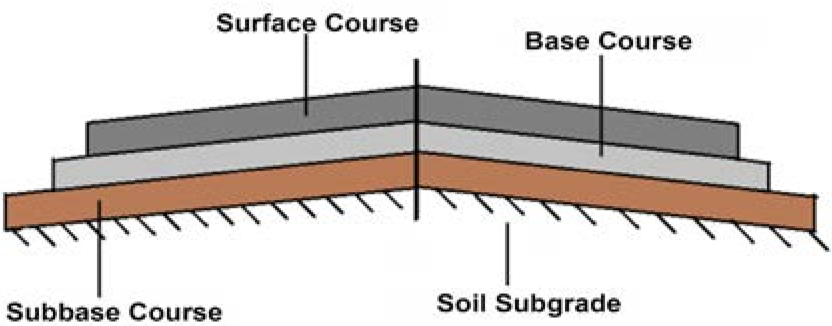


Figure 6.3 Pavement Structure for Low to Medium Traffic



1
2
3
4
5
6
Pavement Evaluation

Pavement structural evaluation involves:

1. Determination of strength properties of the subgrade and pavement layers including the surfacing and the overall pavement strength.
2. Determination of the residual life of the pavement in terms of the remaining number of passes of the equivalent standard axle (ESA) of 80kN (8.160 tonnes) till the pavement fails. This is based on the principle that each passage of the ESA consumes a certain amount of the life of the pavement. The cumulative number of the equivalent standard axle (CESA), which can be carried to the point of failure is the residual life of the existing pavement expressed in million cumulative equivalent standard axles (M CESA). See Section 5.
3. Determination of the pavement design life – the maximum M CESA to failure of the designed pavement. For design purposes, it is required to express the design life in years to enable forecasting and scheduling interventions, which are critical for pavement management, maintenance and rehabilitation.
4. Determination of the structural deficiency of the pavement concerning the forecasted traffic loading.

The structural adequacy of a pavement may be defined as its ability to carry traffic without developing appreciable structural deterioration. It is dependent upon proper construction with suitable material and sufficient thickness to prevent traffic from overstressing the subgrade or any other pavement layers.

Structural evaluation is aimed at determining the current adequacy of the pavement and forecasting its future behaviour under the predicted traffic loading. When a pavement is found to be inadequate, its structural evaluation provides a basis for designing the strengthening required for a given design period.

The strength of the pavement is determined through:

1. Measurement of the bearing capacity and stiffness (from the FWD, Benkelman beam, LWD or similar surveys).
2. Analyses of the characteristics of all pavement layers and subgrade through in-situ tests (DCP, densities, etc.), sampling and laboratory tests, see RDM 5.1 for details.

For any pavement, the strength and subsequently, the structural condition is variable from point to point due to the variable nature of the subgrade and pavement material properties and the lack of uniformity that is inherent in the construction operations.

Therefore, the more damaged a pavement, the more variable its strength characteristics will be, since deterioration is never uniform. Hence, it is almost impossible to define the sections, due to considerable scatter of their characteristics.

This justifies using a statistical approach, based on the analysis of a sufficient number of results. This is referred to as the statistical significance of the data set.

The procedure for structural pavement evaluation involves:

1. Analysis of deflection data from FWD or Benkelman beam, or LWD (see RDM 5.1).
2. Analysis of DCP data (see RDM 5.1).
3. Analysis of Test Pit data and information (see RDM 5.1).
4. Characterisation of pavement layers and materials.
5. Characterisation of subgrade or road foundation.
6. Determination of pavement layer thicknesses (Core logs, DCP and Test pit logs) - see RDM 5.1.
7. Determination of Poison’s ratios and E-moduli of the pavement layers and subgrade.
8. Reporting – results will be used to design pavement maintenance, or rehabilitation as may be required.

6.3.1 Use of the Deflection Measurement

6.3.1.1 Principle and Significance

Passage of a wheel load over a pavement produces a small transient depression on the surface, a deflection. The magnitude of the surface depression or deflection depends on the wheel load, contact area, wheel speed, and the stress-strain characteristics and thicknesses of the various pavement layers and the subgrade.

Therefore, if a standard wheel load, tyre size and pressure, and test procedures are applied, measurement of the surface deflection will enable comparisons between the stiffness of different pavements. It will also provide a means of monitoring the structural strength of pavements over time.

For flexible pavements, the deflection can generally be correlated with the subsequent performance of the pavement under traffic loading. When a transient strain, induced by a wheel load, exceeds a certain critical value in one or more of the pavement layers or the subgrade, it is assumed that a small non-recoverable strain remains. The accumulation of such permanent strains results in the permanent deformation and subsequent cracking of the road surfacing.

High deflections always indicate structural deficiency. However low deflections do not necessarily denote a satisfactory structural condition. The advantages and disadvantages of deflection tests are given in Table 6.1.

Table 6.1 Advantages and Limitations of Deflection Tests

Item No.	Attribute	Advantages	Limitations
1	**Complexity**	Simple, quick and repeatable and can be applied on long stretches of road.	Accuracy is limited when pavement is heavily deteriorated and when pavements were not properly designed but evolved and exhibit high variability.
2	**Statistical significance**	Large quantities of deflection data can be obtained, and greater statistical significance can be achieved.	Data cleansing is necessary, but it is not simple and often overlooked.
3	**Determination of homogeneous sections**	Can be used to determine homogeneous sections required for maintenance and rehabilitation design.	The homogeneous sections may not correspond to the ones derived from pavement materials or pavement structures but strength alone.
4	**Characterisation of pavement layers and materials**	Through back-calculation, they can be used to determine characteristics of pavement layers and materials (i.e., calculation of the moduli and critical stresses and strains)	Can only be used to characterise the pavement layers and materials when key parameters such as the thicknesses and Poisson's ratios are known.
5	**Application**	Appropriate for flexible and semi-rigid pavements which constitute the largest part of the pavements	Not very suitable for rigid pavements because the deflections tend to be very low.

In this respect, it is stressed that deflections measured on thin pavements largely depend on the deformability of the subgrade. It therefore follows that:

1. Low deflections may be measured on inadequate or deteriorated pavement constructed on a strong subgrade.
2. Surface deflections depend on the subgrade strength, particularly, moisture content. Seasonal variations of subgrade moisture reflect by seasonal variations in the deflections. It is then necessary to correlate deflection with the actual subgrade moisture content. It is also essential to measure the maximum deflection corresponding to the subgrade at its wettest (i.e. at the end of a rainy season).

1
2
3
4
5
6
Pavement Evaluation

6.3.1.2 Variation-Characterisation of Homogeneous Sections

Due to many factors affecting the deflections, variations in deflections from point-to-point cause considerable scatter, especially for thin pavements, constructed with natural and heterogeneous materials, and when pavement condition is poorer.

It is then necessary to obtain a sufficient number of readings to enable a meaningful statistical analysis to be carried out. Recommended test intervals are given in Table 6.2.

Table 6.2 Recommended Test Intervals For Deflection Tests

Item No.	Purpose	Advantages
1	**Network management**	200-500
2	**Feasibility**	100-200
3	**Rehabilitation**	50-100
4	**Compliance**	100-200
5	**Premature Failures and Special Investigations**	50-100

The road shall then be divided into homogeneous sections showing similar levels of deterioration and statistical distribution of deflection, similar pavement structures, subgrade type, surface condition and traffic loading.

6.3.1.3 Determination of Uniform Sections

Determine uniform sections using the following method:

1. Collect the maximum deflections and chainages obtained during the surveys (RDM 5.1)
2. Use the CUMSUMS formula (Equation 6.2) to determine uniform sections based on the maximum deflections (d_o). Alternatively, use d_{90}, which is the 90% confidence value of the maximum deflections (d_0) in the formula:

$$S_i = (d_{90i} - d_{90m}) + S_i$$

Equation 6.2

Where,

d_{90i} = Data value at chainage i.

d_{90m} = Mean value.

S_i = Cumulative sum of the deviations from the mean value at chainage i.

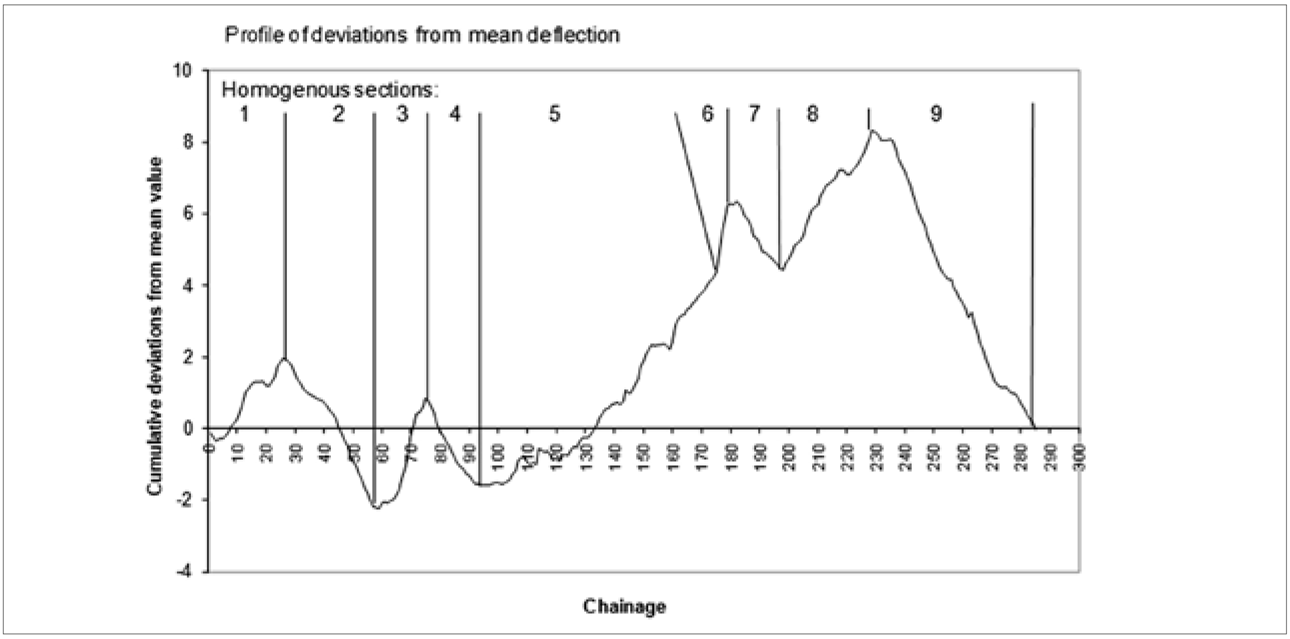
3. Create a Table 6.3 illustrated below for the calculation of CUMSUMS:

Table 6.3 Tabulation of Data for Calculation of CUMSUMS

Chainage (km)	Deflection Measurement at Chainage, d_{90i} (micrometres)	S_i-1	S_i

4. Plot S_i against chainages. The plot will be similar to the graph in Figure 6.4. The uniform gradients indicate uniform sections.

Figure 6.4 Illustration of Determination of Uniform Sections From S_i Vs Chainage Plot



5. Determine coefficients of variations for a more accurate determination of uniform sections (Equation 6.3).

$$Coefficient\ of\ Variation\ (CoV) = \frac{Standard\ Deviation\ (S)}{Mean\ (X_m)}$$

Equation 6.3

$CoV < 0.20$ = good homogeneity.
$0.2 < CoV < 0.3$ = moderate homogeneity.
$CoV > 0.3$ = poor homogeneity.
Consider the section as uniform when $CoV \leq 0.25$.

6. Determine representative values for the uniform sections, which should be considered as the 90% confidence value using Equation 6.4.

$$Representative\ value = X_m + 1.3\ S$$

Equation 6.4

This equation represents 90% reliability or confidence.
Therefore, for deflections, use Equation 6.5:

$$Representative\ value,\ d_{90} = d_m + 1.3\ S$$

Equation 6.5

Where,
d_m = mean deflection.
S = standard deviation.

The method should be applied to any other design parameters.

1
2
3
4
5
6
Pavement Evaluation

6.3.1.4 Data Cleansing and Outliers

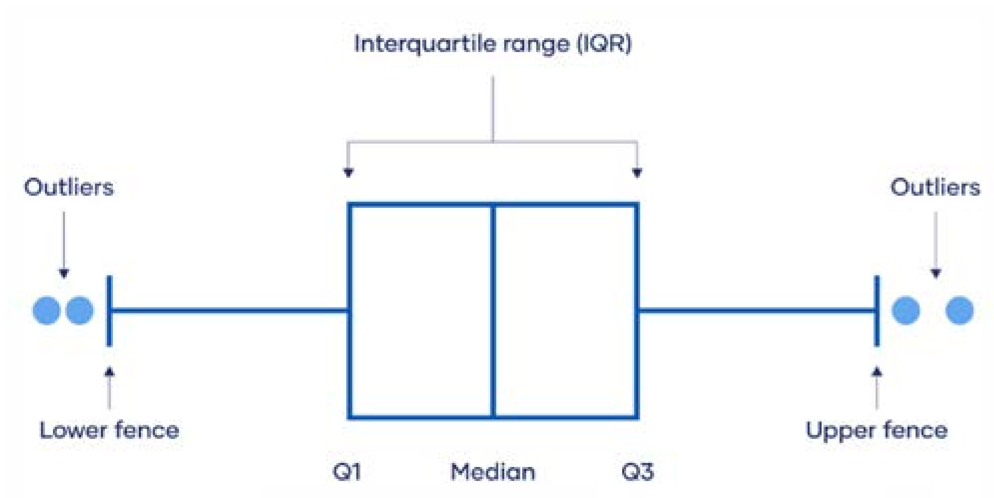
Should the variations in the data be high as indicated by the CoV it may be necessary to carry out data cleansing and the outliers shall be determined statistically. Outliers are not simply considered as erroneous data. The outliers could be caused by localised weak spots or a small section of the pavement exhibiting peculiar results. These could help the engineer to design for localise maintenance or repairs.

Use the quartile method:

1. Determine upper and lower boundaries using the method of quartiles, see Figure 6.5.
2. Determine the values outside the upper and lower boundaries, referred to as outliers.
3. Determine whether the outliers are 'true' or 'false'. 'True' means they represent possible reality judging from the engineering understanding (engineering judgment) or 'false' which from engineering judgment would be impossible and they are therefore an error of measurement.
4. Include the true ones if they do not distort the trends or relationships by exaggerating some parameters. This would be the case where representative values, for a given section need to be calculated or trend lines need to be drawn.
5. Also, look for data inconsistency, e.g., deflections should decrease with increases in radial distance from the central load. If this trend is not fulfilled, then the data out of the trend are erroneous and should be discarded. If the error repeats itself for subsequent test points or many points exist in the data set, there is something wrong with the equipment and the deflection tests should be repeated after solving the problem.
6. If the deflection values of the last geophone i.e., 1800 mm or 2100 mm from central load are below 10 micrometres then the result should be discarded.
7. The severity of cracking should be checked to allow for appropriate result adjustments. Results may be discarded if the cracking is excessive.

The method of quartiles:

Figure 6.5 Determination of Data Quartiles and Outliers



1. During data collation write the data in increasing order from the smallest to the largest value.
2. Determine the first quartile Q1, the second Quartile Q2 (= median value), 3rd quartile Q3 and 4th Quartile Q4. Q1 = the median value of all values below the median of the full data set (Q2), and Q3 = the median value of all the values above Q2.
3. Determine the Interquartile Range IQR = Q3 - Q1.

4. Upper boundary = Q3 + (1.5 x IQR).
5. Lower boundary = Q1 – (1.5 x IQR).
6. Values outside these boundaries are the outliers.

Ideally, this procedure should be carried out as part of RDM 5.1. However, it has been provided here because further data cleansing, determination of outliers and representative values of sections are essential during pavement evaluation.

It is essential to study the locations of all anomalous results and all apparent sections of uniform condition to identify if there are any obvious reasons and common factors.

6.3.1.5 Representative Deflection

Experience has shown that the distribution of deflections in a homogeneous section approximates a Normal or Gaussian distribution. Such a distribution will confirm the homogeneity of the chosen section. The characteristic or representative deflection $D90$ is calculated using Equation 6.6:

$$d_{90} = d_m + 1.3s$$

Equation 6.6

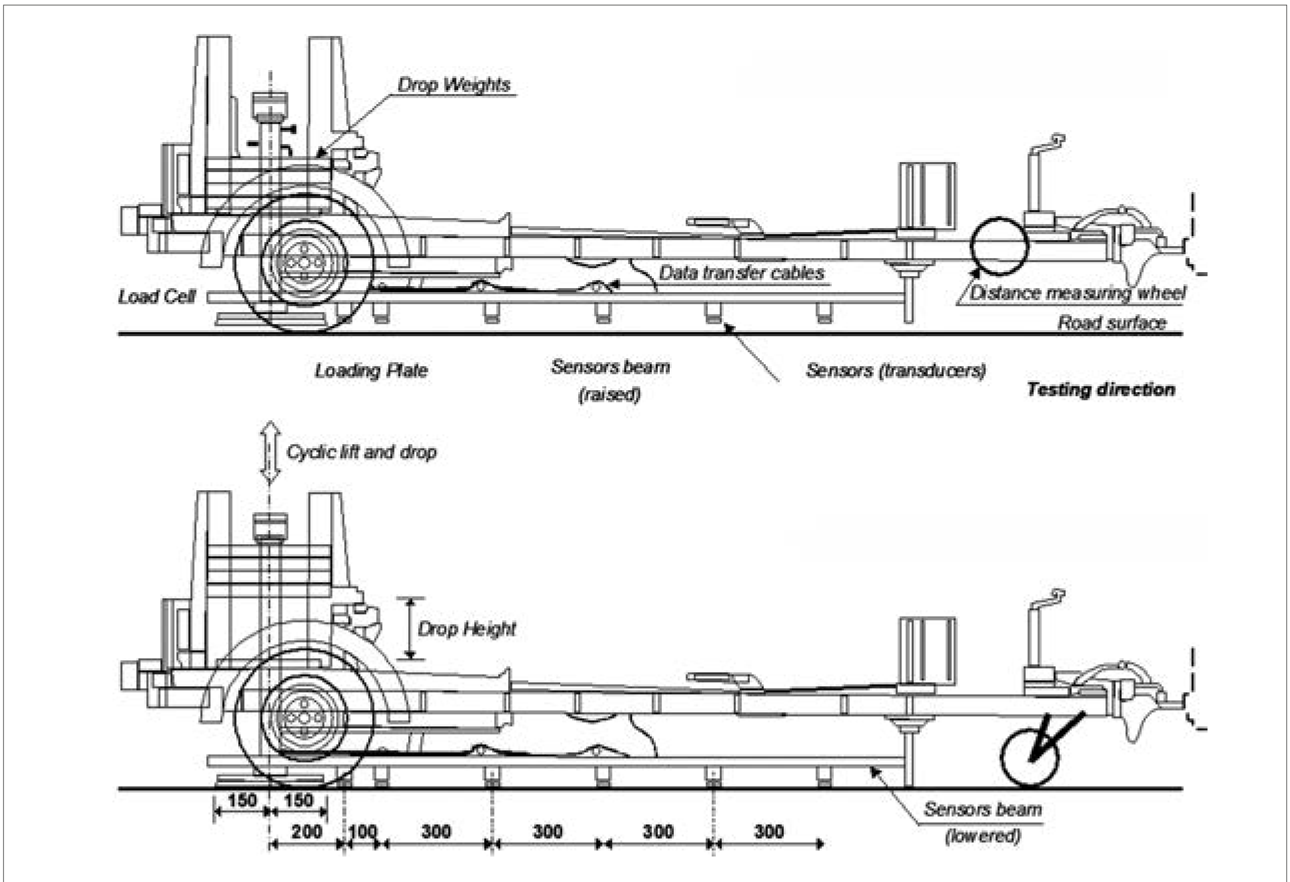
Where 'd_m' is the mean deflection and 's' is the standard deviation.

Thus, 10% of the deflections measured are higher than D_{90}, this guarantees the design of the overlay or strengthening within 90% confidence limits.

The deflections are carried out using procedures given in Volume 5.1. The FWD is more commonly used and is presented in Figure 6.6. A Benkelman beam and a lightweight deflectometer are used to measure deflections. Details of deflection test methods using the FWD, Benkelman beam and LWD are given in RDM 5.1.

The critical features are shown in Figure 6.6.

Figure 6.6 Configuration of the FWD in Both the Transportation and Testing Modes



The deflections measured by the different sensors can be interpreted to indicate the relative strengths of the different layers of the pavement and the subgrade. The deflections at various points are used to determine various indices including:

1. Surfacing index.
2. Base index.
3. Subbase index.
4. Subgrade index.
5. Radius of curvature.
6. Zone of influence.

The deflection measurements from all the geophones D_0 to D_{1800} or D_{2100} are used to determine the deflection bowl. A shallower bowl depicts higher pavement strength, and a deeper bowl depicts weaker pavement. Also required are the thicknesses of the pavement layers including the selected subgrade or foundation. Software packages apply the pavement structure, layer thicknesses, Poison's ratios and the deflections to compute the layer stiffnesses in the form of the Elastic Moduli (E-moduli), which give the relationship between the stress and strain induced on each layer under load (wheel load). Any material when subjected to stress will undergo some form of strain. The elastic modulus, which is the stiffness of the material or element, is given by the general relationship, Equation 6.7.

$$E - modulus = \frac{Stress}{Strain}$$

Equation 6.7

Details of these approaches and the procedures involved in the analysis of the structural condition of the pavement are given below. The evaluation process will develop parameters required in overlay design.

6.3.2 Use of Test Pits

Data and information collected from test/trial pits are critical in pavement evaluation. Refer to RDM 5.1 for the procedure for test/trial pit excavation, logging, sampling of materials, measurement of in-situ densities and moisture, etc. for details. The reports on test/trial pits should provide the following information:

1. Pavement layer thicknesses.
2. Characterisation of the different pavement layers and subgrade – in terms of the types and properties of materials. Many material types of can be encountered in a pavement structure and their inherent variability can influence the results of the pavement evaluation.
3. Condition of the layers and subgrade level of deterioration and moisture condition.

6.3.3 Characterisation of Pavement Materials

Table 6.4 provides a summary of information that characterises materials for maintenance and rehabilitation design. For more details on pavement materials refer to RDM 3.2, RDM 3.3 and RDM 3.4.

Table 6.4 Characterisation of Materials for Maintenance and Rehabilitation Design (RDM 3)

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 soaked 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 soaked 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 soaked 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 soaked 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 soaked 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 soaked 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 soaked 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 soaked 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 soaked CBR of 45 %	• Capping on subgrade classes S3 to S5 for foundation classes F2 to F4	GM9
G50	Natural gravels or natural materials blended with up to 30 % stone aggregates of minimum 4 days soaked CBR of 50 %; OR crushed stone gravel, crusher run of indeterminate CBR but complying with the gradation	• Base for Traffic classes TC0.5 and TC0.25.	GM10
G80	Natural gravels or natural materials blended with up to 30 % stone aggregates of minimum 4 days soaked 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.	GM11

This table continues on the next page...

Code	Material (%)	Application	Chart
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.	GM12
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.	
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.	
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.	
HPS	Hand Packed Stone of maximum ACV 35 % and LAA 50 %. Mostly fresh trachytes, soft stone, basalts, and other igneous rocks.	• Sub-base and Base for traffic classes TC0.25 to TC10.	GM13
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	GM14
Hydraulically Improved Materials and Hydraulically Bound Materials			
HIG50	Lime and hydraulically improved granular material of minimum CBR of 50 % after 7 days cure & 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 & 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 & 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 & 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

Code	Material (%)	Application	Chart
HMS1	Hydraulically modified stone of minimum UCS 1.2 MPa and maximum UCS 2.5 MPa after 7-day cure & 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 & 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 & 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 & 7-day soak	• Base for traffic class TC50 and higher. • Sub-base for concrete pavement for traffic class TC150 and higher.	HB3
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
BSM175	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. Enrobé à Module Élevé (EME), where low penetration very hard, not brittle bitumen binders are used to produce very stiff asphalt layers.	• 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

Code	Material (%)	Application	Chart
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

6.3.4 Characterisation of Pavements

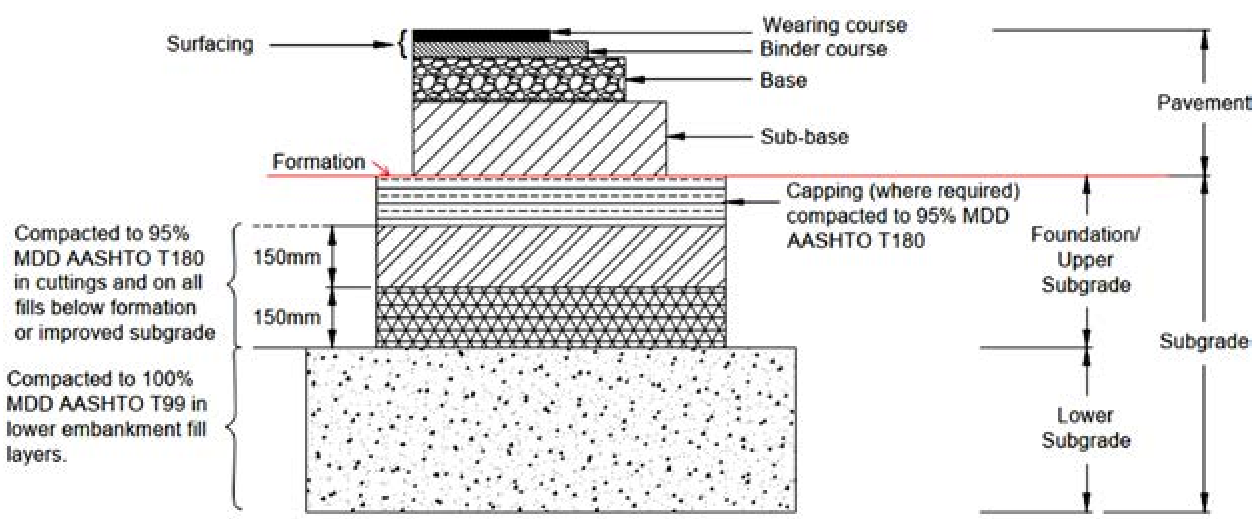
Pavement layers are made of different materials resulting in layers with different characteristics. Understanding these characteristics is important in subsequent stages of pavement evaluation. Key parameters include:

1. Classification of materials
2. In-situ E-moduli of the layers
3. Moduli of rupture of concrete and cement-stabilised materials
4. Poison’s ratios
5. Strength of materials
6. Bound, unbound and modified materials.

6.3.4.1 Characteristic Pavement Structure Based on Foundation Design Approach

A characteristic pavement structure given in Figure 6.7 shows the terminology used for the overall pavement structure and individual pavement layers.

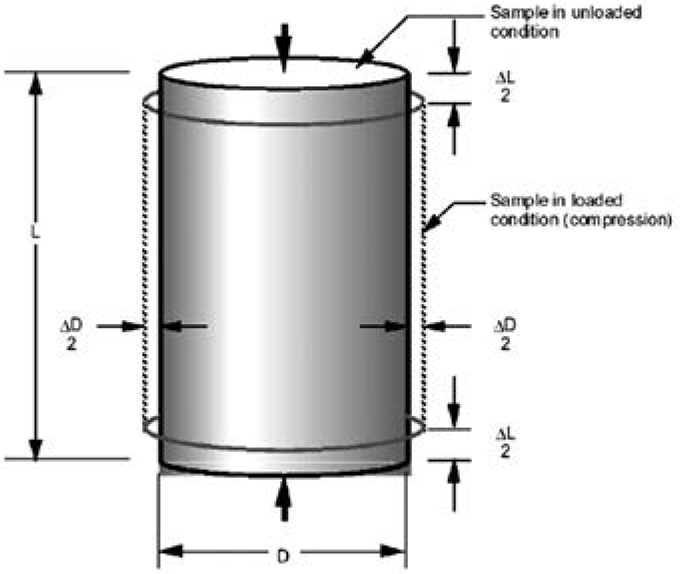
Figure 6.7 Characteristic Pavement Structure



6.3.4.2 E- Moduli and Poison's Ratios of Pavement Structural Layers

Poisson's ratio (μ) is the relationship between the strain in the direction of the load and the strain that occurs perpendicular to the load or radially, Figure 6.8.

Figure 6.8 Illustration of the Determination of the Poisson's Ratio (AASHTO)



$$\mu = -\frac{\epsilon_D}{\epsilon_L}$$

Where

$\mu =$ Poisson's ratio

$\epsilon_D = \frac{\Delta D}{D}$ = strain along the diametrical (horizontal) axis

$\epsilon_L = \frac{\Delta L}{L}$ = strain along the longitudinal (vertical) axis

Poisson's ratio is used in calculating the strain that occurs in the 3 dimensions of a material, and in this case, the pavement layers or subgrade, when a load is applied on the surface. E-moduli and Poisson's ratios for new pavement materials are given in Table 6.5, Table 6.6, Table 6.7 and Table 6.8. For more details refer to RDM 3.4.

Table 6.5 Subgrade Moduli and Poisson's Ratio

Soil Class	Ranges of Subgrade CBR and their median values	Modulus	Poisson's ratio
S1	2 - 5 ($\bar{x}$ ≥ 3.5%)	40	0.45
S2	5 - 10 ($\bar{x}$ ≥ 7.5%)	65	0.45
S3	7 - 13 ($\bar{x}$ ≥ 10%)	75	0.45
S4	10 - 18 ($\bar{x}$ ≥ 14%)	95	0.45
S5	15 - 30 ($\bar{x}$ ≥ 22.5%)	130	0.45
S6	≥ 45%	200	0.35

Table 6.6 Capping and Sub-base Materials Moduli and Poisson's Ratio

Material	Elastic modulus (MPa)	Poisson's ratio
Natural material **(G25)**	130	0.45
Natural material **(G30)**	150	0.45
Natural material **(G45)**	200	0.35
Hydraulically improved granular material **(HIG 50)**	200	0.35
Hydraulically improved granular material **(HIG60)**	300	0.35
Graded crushed stone on subgrade class S3 and S4	300	0.35
Graded crushed stone on subgrade class S5: 400 MPa	400	0.35
Graded crushed stone on subgrade class S6 and above	500	0.35
Bitumen Stabilised Material **(BSM50)**	400	0.35
Bitumen Stabilised Material **(BSM100)**	1500	0.35

Table 6.7 Base Materials Moduli and Poisson's Ratio

Material	Elastic modulus (MPa)	Poisson's ratio
Natural gravel **(G50)**	300	0.35
Natural gravel **(G80)**	300	0.35
Hydraulically improved granular material **(HIG 50)**	200	0.35
Hydraulically improved granular material **(HIG60)**	300	0.35
Hydraulically improved granular material **(HIG100)**	450	0.30
Hydraulically improved granular material **(HIG160)**	1000	0.25
Hydraulically modified stone **(HMS1)**	1300	0.25
Hydraulically bound stone **(HBS3)**	4000	0.25
Hydraulically bound stone **(HBS6)**	7000	0.25
Hydraulically bound stone **(HBS9)**	10000	0.20
Graded crushed stone	400	0.35
Dry bound or Wet-bound Macadam	600	0.35
Hand Packed-Stone	350	0.35
Bitumen Stabilised Material **(BSM50)**	400	0.35
Bitumen Stabilised Material **(BSM100)**	1500	0.35
Bitumen Stabilised Material **(BSM175)**	2000	0.35
Sand bitumen mix	1000	0.35
Dense bitumen macadam **(DBM)**	5000	0.35
Superpave DBM	5000	0.35
EME	8000	0.30

Table 6.8 Surfacing Materials Moduli and Poisson's Ratio

Material	Elastic modulus (MPa)	Poisson's ratio
Asphalt Concrete Type I (High Stability)	4000	0.35
Superpave Asphalt Concrete	4000	0.35
Stone Mastic Asphalt	5000	0.35
Asphalt Concrete Type II (Flexible), Sand Asphalt and Gap Graded Asphalt	2500	0.35

6.3.4.3 Specifications for Minimum Foundation/Subgrade Strength

The surface moduli and CBR strengths are important for determining the resilience of the subgrade and the road foundation (see Chapter 9) for the calculation of surface modulus. The surface moduli and CBR are also used to characterise the strength of pavement layers and in the mechanistic-empirical design in Section 9.6 and the AASHTO Structural Number Method in Section 9.7. The characteristic specifications for minimum foundation or subgrade surface moduli and CBRs are given in Table 6.9. More details on foundation classes are given in RDM 3.3 and RDM 3.4.

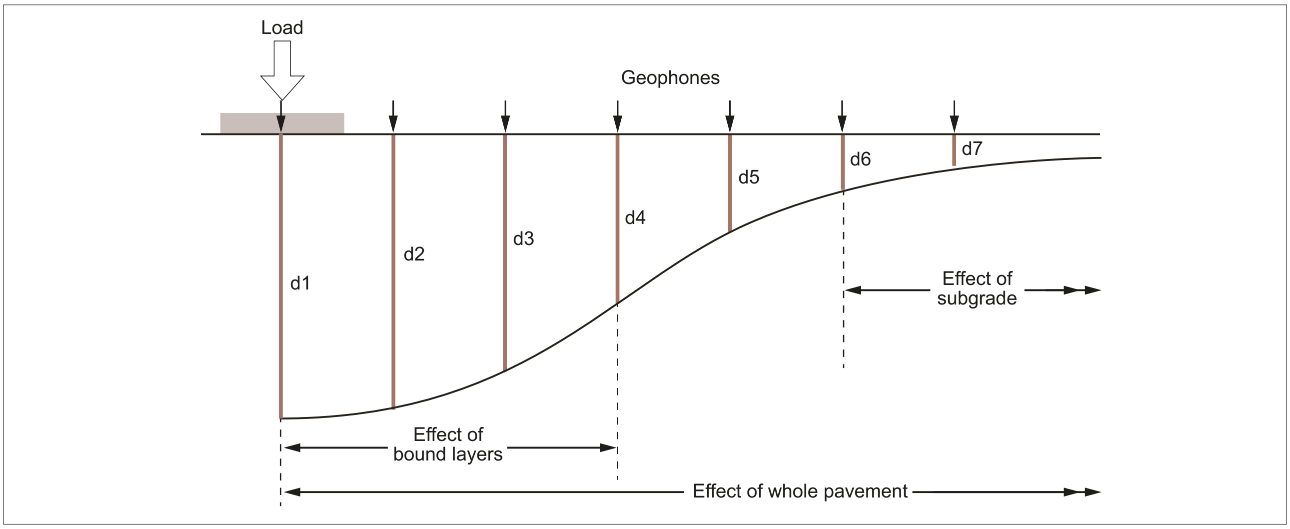
Table 6.9 Pavement Foundation Classes Based on Minimum Surface Moduli and CBR

Foundation Class	Minimum Foundation Requirement		Equivalent Subgrade Class	Elastic modulus (MPa)	Poisson's ratio
	Surface Modulus (MPa)	Equivalent Minimum CBR (%)			
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		

6.3.4.4 Back Calculating E-Moduli of Pavement Layers and Subgrade/Foundation

Deflections occur on the pavement when a load impulse is applied using a FWD, LWD, or a static load by Benkelman beam. The highest deflection is at the centre of the load and they become progressively less with an increase in the radial distance. This is illustrated in Figure 6.9.

Figure 6.9 Illustration of FWD Deflection Bowl



1
2
3
4
5
6
Pavement Evaluation

A deflection bowl can be plotted for each drop carried out at each point. However, averages of the deflection measurements from the 3 drops carried out at each point tested are used.

Deflection can also be determined theoretically using the stress-strain relationships based on the Burmeister Theory. This method considers a multilayer elastic system. When a load is applied on top of the elastic layer, elastic deformation occurs which is the strain ($\mathcal{E}$) under a given stress (σ). The E-modulus of the layer determines the amount of strain that occurs in the layer under load. Using the multi-layer elastic theory, given the layer thicknesses ($h1 \ldots hi$) and the E-moduli and Poisson's ratios, the theoretical deflections can be determined.

Back calculation

The back-calculation method involves the determination of the E-moduli of the layers from the deflections obtained in the field. In this method:

1. The unknowns are the E-moduli of the pavement layers and the subgrade that need to be determined.
2. Layer thicknesses are obtained from the test pit logs.
3. Reference deflections are the FWD deflections from the field tests.
4. Poisson's ratios for the different materials as new are given in Table 6.5, Table 6.6, Table 6.7 and Table 6.8.
5. Table 9.1 provides a range of E-moduli for different pavement materials existing on the road. The ranges indicate the E-moduli of materials in fair to good condition (i.e., the higher values), and for deteriorated materials at the lower end and any in-between depending on the deterioration level of the layer.
6. **Procedure:** Back calculating the E-moduli of the pavement layers is carried out through an iterative process using assumed moduli within a given range and calculating the theoretical deflection. The theoretical deflections are then compared with the deflections obtained in the field. The iterations are continued until the difference between the theoretical and field deflections is small i.e., a close match is obtained. The E-moduli that give the close match are then taken as the E-moduli of the pavement layers and subgrade. While the iterative process can be carried out manually, it is cumbersome. Several software packages have been developed, which can carry out the iteration on the same basic principle but in a relatively short time.
   a. Input details of the project into the software interface.
   b. Upload the deflection data as a file or as the average values depending on the requirements of the software requirements.
   c. Enter layer types (wearing course -AC, base, subbase, subgrade/foundation)
   d. Enter the material types:
      i. Bound – AC, DBM, Hydraulically Bound Stone, HBM, HBS, etc.
      ii. Unbound – GCS, HIB, HISB, G, SG.
   e. Enter the layer thicknesses in the designated cells.
   f. Enter the Poisson's ratios for the materials.
   g. Enter the E-modulus ranges for each pavement layer and subgrade within the recommended ranges given in Table 9.1.
   h. Make an initial estimation of the moduli within the ranges given.
   i. Follow the other software requirements, which may include the method of analysis and other data, or information required such as the temperatures, seasonal factors, etc.
   j. Start the iterations after specifying the maximum allowable error.

k. Once the iteration is completed, check the resultant E-moduli for the layers, and the number of iterations carried out by the software; the maximum error indicates the closeness between the actual and theoretical deflection bowls. The value is calculated automatically but based on the Root Mean Square Error (RMSE) equation given in Equation 6.8:

$$\text{RMSE} = \sqrt{\sum_{1}^{nd}\left[\frac{1}{n_d}\left(\frac{d_{ci}-d_{mi}}{d_{mi}}\right)^2\right]}$$

Equation 6.8

RMSE should be between 1% and 2%

7. Record the resultant E-moduli characterising the in-situ strength of the pavement layers and the subgrade. See Sections 6.3.3 and 6.3.4 for more details.
8. The subgrade modulus can also be calculated from deflections using Equations 6.9 to Equation 6.11:

Using deflection at radial distances for d_{1200}, d_{1500}, d_{1800}.

$$E_{SG} = \frac{(1 - \mu^2)\text{ x } P}{3.14 \text{ x } r \text{ x } d}$$

Equation 6.9

Where,

ESG = the Elastic modulus of subgrade, see Section 7.6.2 for more details.

P = load (N).

R = average radial distance.

μ = Poisson's ratio.

d = average deflection for d_{1200}, d_{1500} and d_{1800}.

Consider the range of ESG as given below:

$$Lower\ bound = 1.2 \text{ x } (E_{SG})^{0.8}$$

Equation 6.10

$$Upper\ bound = 1.2 \text{ x } (E_{SG})^{1.2}$$

Equation 6.11

6.3.4.5 Pavement Strength from DCP Tests

The procedures for the test are given in RDM 5.1.

The report from the field investigation will include:

1. Penetration data.
2. Type and condition of the materials.
3. Moisture content of the materials (from moisture content obtained from laboratory tests on the samples obtained soon after testing).
4. Layer boundaries, thicknesses and structure of the pavement.
5. CBRs.
6. Structural Numbers (SN).

The CBRs are calculated from the correlations between the laboratory CBRs and the DCP penetration rates. Some of the correlation equations that are used internationally are given in Appendix D. They include the:

1. TRL Equation and the Kleyn Equation for the 60-degree cone
2. Smith and Pratt Equation, and Van Vuuren Equation for the 30-degree cone.

1
2
3
4
5
6
Pavement Evaluation

The Structural Number (SN) for the pavement layers is given by Equation 6.12:

$$SN = 0.0394xaxt$$

Equation 6.12

Where,

a = layer strength coefficient.

t = thickness of the layer (mm).

Layer strength coefficients are generally calculated using Equation 6.13 to Equation 6.15:

a. Granular unbound base materials:

$$a_i = 29.14CBR - 0.1977CBR^2 + 0.00045CBR^3 \ \ x10^{-4}$$

Equation 6.13

b. Cemented bases:

$$a_i = 0.075 + 0.039UCS - 0.00088UCS^2$$

Equation 6.14

c. Unbound subbases:

$$a_i = 0.0075 + 0.184log_{10}\ CBR - 0.0444(log_{10}\ CBR)^2$$

Equation 6.15

The results of the analyses will provide the parameters necessary to characterise the pavement layers and materials. These parameters are used at the design stage, see Chapter 9.

6.3.4.6 Pavement Layer Thickness

Pavement layer thicknesses and their depth from the surface of the pavement:

1. When evaluating the pavement, it is important to determine the representative thickness for each uniform section, hence each uniform section must have 1 or more test pits depending on the length of the section. Pavement and layer thicknesses shall be provided in mm.
2. Complications may occur where layers may have not been engineered but evolved resulting in high variability. Lack of homogeneity may have a significant impact and should be considered in the analysis and design.
3. Test pits should be excavated on points exhibiting the highest deflections to ensure that the worst-case scenarios are evaluated. The designer should consider this during pavement evaluation.

6.3.5 The Use of Radius of Curvature Measurements

6.3.5.1 Principle and Significance

The measurement of deflections and calculation of the radii of curvature for the Benkelman Beam, FWD and LWD are detailed in RDM 5.1.

The zone of influence or radius of curvature of the deflected road surface is a better indicator of the strains imposed on the pavement. Indeed, the radius of curvature of the deflection basin is largely governed by the rigidity of the upper pavement layers.

Experience has confirmed that pavement performance and condition are more closely related to the severity of bending than deflections. On rigid and semi-rigid pavements, the magnitude of surface deflection has little significance, and the main structural indicator is the radius of curvature.

RDC measurements must be incorporated in all deflection survey work to provide a complete assessment and enhance the deductions regarding the pavement condition and the strengthening required.

High radii of curvature always indicate a rigid base and surfacing, whereas low radii of curvature correspond to an unbound pavement. An 'unbound' layer consists of either flexible material or broken rigid material.

6.3.5.2 Advantages

Deflection measurement alone cannot account for a pavement's structural behaviour. Measurements of radii of curvature are a meaningful and reliable means of supplementing deflection measurements and enabling a more rational evaluation of the structural condition of a pavement.

6.3.5.3 Variation-Characterisation of Homogenous Sections

The radius of curvature measurements always exhibits appreciable scatter, especially on deteriorated pavements. It is therefore necessary to obtain a sufficient number of values to allow for a meaningful statistical analysis. For the Benkelman beam, it is recommended that radii of curvature be measured simultaneously with the deflections. This has the advantage of providing a continual evaluation of the pavement’s structural condition. For FWD and LWD the setup of the geophones allows for radii of curvature to be calculated.

Frequency curves for the radii of curvature do not fit the symmetrical bell-shaped normal distribution curve. However, the distribution of logarithmic values of the radii of curvature approximates to Gaussian distribution. The characteristic radii of curvature (R_{10}) are then calculated using Equation 6.16:

$$logR_{10} = m[logR\ ] - 1.3S[log\ R]$$

Equation 6.16

Where, m ($log\ R$) is the mean of $Log\ R$ and s($log\ R$) is the standard deviation of log R.

6.3.5.4 Standard Procedure for Radius of Curvature Measurement

The radius of curvature when measured with a Benkelman beam although generally satisfactory, is not very accurate. The accuracy and reproducibility of the measurement can be greatly improved by the use of an X/Y recorder, which can be connected to the deflection beam. This electronic device automatically records the deflection bowl.

6.3.6 Drainage Analysis

To properly evaluate the structural condition of an existing pavement, it is necessary to know the strength properties of the subgrade and their possible seasonal variations. In this respect, it is equally important to assess the condition of the drainage system.

1. For a routine survey or feasibility study, the use of design and construction records, combined with a visual assessment of the nature of in-situ soils and the adequacy of drainage, should be sufficient to define homogeneous sections in terms of subgrade strength.
2. For the final design of a strengthening project, it is recommended that the following procedure be adopted.
    a. Ideally, both deflection and test pits should be carried out at the end of the rainy season.
    b. In this first stage, the adequacy and the condition of the drainage system should be thoroughly assessed (obviously, during a rainy period).
    c. This should cover both internal drainage of the pavement (crossfalls, surface impermeability, shoulders, drainage layers etc.) and external drainage (ditches, drains, pipes, culverts etc.).
    d. The possible influence of groundwater should also be evaluated. If a water table exists near the formation, its characteristics (level, flow, seasonal variations, etc.) should be thoroughly studied. If springs are noticed in cuts, their possible effects on subgrade (and pavement) strength should be assessed.
    e. This will facilitate the division of the road into homogeneous sections based on drainage conditions. Localised spots where peculiar drainage problems occur should be recorded and treated individually.

6.3.7 Existing Pavement Structure Analysis

It is essential that all pavement layers are accurately identified, and their condition appraised so that the residual strength of the existing pavement can be evaluated and the causes of structural deficiency and surface distress determined.

1. For routine survey or feasibility study, used:
    a. Original design and construction records.
    b. Surface condition survey data.
    c. Define homogeneous sections.

If construction records are not available, or not accurate enough, sufficient holes shall be excavated and limited testing carried out to ascertain the thickness, type and state of the various layers.

2. For the final design of a strengthening project, the following procedure is recommended:
    a. Excavate at least 2 test pits at points of highest deflections.
    b. Measure the thickness of each pavement layer.
    c. Visually identify and describe the material for each layer.
    d. Determine moisture content and density for bound materials, and take cores for further tests in the lab.
    e. Determine homogeneous sections, in terms of material.

It is essential that all test pits are properly backfilled, compacted and resurfaced upon completion and that they do not create points of future failure in the pavement or safety hazard.

More details on the excavation of test pits and collection samples and measurements required are given in RDM 5.1. The field investigation report will provide information on the following:

1. Untreated or Improved material.
    a. Measurements of Field Density.
    b. Field Moisture Content (FMC).
    c. One sample shall be taken and shall be subjected to the following tests:
        i. Particle size analysis.
        ii. Atterberg limits.
        iii. Compaction (vibrating hammer for graded crushed stone, Modified AASHO for other materials).
        iv. CBR (natural material).
        v. Los Angeles Abrasion test (stone).
        vi. Aggregate Crushing Value test (stone).
2. Stabilised (Bound) Material.
    a. At each point, 6 undisturbed samples should be obtained by core drilling. This applies to all treated layers having sufficient cohesion (whether treated with cement, lime or bitumen).
    b. The following measurements should be made:
        i. On 3 core samples: - Density, Unconfined Compressive Strength, and Moisture Content.
        ii. On 3 others: - Density, Tensile Strength (split test), and Moisture Content.

6.3.8 Shoulder Assessment

The type of shoulders should be noted and their condition should be assessed visually. If it appears that there are problems involving the shoulders (drainage of pavement layers, edge restraint, etc.), specific investigations will be required.

In particular, where substantial works affecting the shoulders are planned, e.g. pavement widening, shoulder upgrading, and placing of a drainage layer, the characteristics of the existing shoulder material should be determined, with special regard to their possible re-use or re-cycling. The thickness and the volume of existing shoulders should also be measured. It may be necessary to dig holes through the shoulders, to measure thicknesses and obtain samples.

7 Criteria for Maintenance and Rehabilitation

7.1 General

Continuous and periodic evaluation of the pavement condition is necessary to determine the trend of changes and to plan and design corrective measures.

The definition of unsatisfactory service in a pavement structure is a complex problem, since it depends, among other factors, on the level of service, safety and the rate of maintenance considered normal or acceptable. Therefore, the criteria suggested in this section should be regarded only as indicative.

7.2 Surface Condition Criteria

Depending on the extent of cracking and transverse deformation, pavements may be classified according to the indices in Table 7.1 (and RDM 5.1).

Table 7.1 Surface Condition Criteria

Rutting under a 3-m straightedge or profilometer		Degree of cracking (Visible cracks)		Criteria for Rehabilitation/ Maintenance
Rut depth Index (RdI)	**Deformation or Rut depth** (mm)	**Cracking Index** (CrI)	**Crack width**	**Intervention**
1	< 10	1	<1	Maintenance
4	10 – 15	4	1-2	Maintenance
9	15-20	9	2-3	Maintenance
16	20-25	16	>3	Rehabilitation
25	> 25	25	> 3 + spalling	Rehabilitation

The following points should be noted:

1. With a normal cross fall of 2.5%, ponding will generally occur at rut depths greater than 12 mm.
2. For flexible pavements, when the rut depth in the wheel path, reaches about 20 mm, cracking will occur and water will penetrate the pavement, causing rapid deterioration.
3. When the degree of cracking reaches about 3 mm, pothole development is imminent and immediate maintenance is required.

Consideration should also be given to other types of surface distress, such as ravelling, stripping, peeling etc. The surface percentage covered by patched areas should also be measured since patching represents incurred deterioration. The following criteria for road deterioration should be applied:

1. Road Requires Local Patching when one of the following conditions arises:
   a. Rut depth exceeds 20 mm / RdI > 4.
   b. Cracking exceeds 2 mm/ CrI > 4.
2. Road Requires Resealing (surface dressing or slurry seal) when any one of the following conditions arises: -
   a. Mean cracking in the wheel path exceeds 2 mm and CrI>9.
   b. General deterioration of the surface (fine crazing, ravelling, stripping, etc.) covers more than 20 % of the carriageway area over the length showing distress.
3. Road Requires Minor Overlay or Resurfacing when any of the following conditions arise: -
   a. Cracking covers more than 30 % of the wheel paths over the length showing distress for trunk roads and 50 % for other roads or (CrI > 9).
   b. Rut depths in nearside traffic lanes reach 15 mm over more than 20% of the length showing deformation for trunk roads (RdI > 9).

1 2 3 4 5 6 7 Criteria for Maintenance and Rehabilitation

4. Road Requires Major Overlay or Reconstruction when the mean rut depth in either wheel path exceeds 20 mm for trunk roads and 25 mm for other roads or (CrI > 16 or RdI > 16).

7.2.1 Surface Roughness Criteria

1. **Road Requires Resurfacing** when surface irregularity measured by a bump integrator exceeds the following values:
   a. Trunk roads IRI ≥ 2,800 mm/km.
   b. Other roads IRI ≥ 3,100 mm/km.
2. **Road Requires Overlay or Reconstruction** when the surface irregularity exceeds the following values:
   a. Trunk roads IRI ≥ 3,400 mm/km.
   b. Other roads IRI ≥ 3,750 mm/km.

7.2.2 Present Serviceability Index Criteria

1. **Road Requires Rehabilitation** when Present Serviceability Values (PSI) exceed the following values:
   a. Trunk roads, PSI ≤ 2.5.
   b. Other roads, PSI ≤ 2.0.
2. Plan Rehabilitation when Present Serviceability Values (PSI) exceed the following values:
   a. Trunk roads, PSI ≤ 3.0.
   b. Other roads, PSI ≤ 2.5.

7.2.3 Present Serviceability Rating

The determination of the present serviceability (PSR) rating is given in RDM 5.1. The criteria for PSR are given in Table 7.2.

Table 7.2 Present Serviceability Rating (PSR)

PSR	Rating Description	Intervention
0-1	Very Poor	Rehabilitation/ reconstruction
1-2	Poor	Rehabilitation
2-3	Fair	Periodic maintenance
3-4	Good	Routine maintenance
4-5	Very Good	Routine maintenance

7.2.4 Pavement Condition Index (PCI)

The surface condition rating is based on the criteria given in Table 7.3.

The PCI is derived from calculating of indices using the extent and severity of the defects. Details are covered in RDM 5.1

$$Index = severity \text{ x } extent$$

The pavement condition index is derived from the indices by subtracting the surface condition indices (expressed as a percentage) from 100 % (representing a new pavement).

The surface condition rating and pavement condition index are calculated using Equation 7.1, Equation 7.2 and Equation 7.3.

$$PCI_i = PCI_{max} - \sum_1^k Deduct\ (SCR)$$

Equation 7.1

$$SCR = \left[\sum_1^k \frac{(I_1 + I_2 + I_3 + \ldots + I_k)/k}{25}\right] \times 100$$

Equation 7.2

$$PCI = PCI_{max} - \left[\sum_1^k \frac{(I_1 + I_2 + I_3 + \ldots + I_k)/k}{25}\right] \times 100$$

Equation 7.3

Where,

PCI = Pavement Condition Index, %.

PCI_{max} = Maximum PCI = 100 % (new pavement).

SCR = Surface condition rating (100 % for fully deteriorated pavement and zero for a new pavement.

I = The defect index (RdI, CrI, PPI, etc).
The maximum for each defect is 25, severity 5 x extent 5 = 25).

K = Number of defect types considered.

Table 7.3 Pavement Condition Index

Rating	PCI (%)	Intervention
5	80 – 100	Routine maintenance
4	60 – 80	Routine maintenance and surface treatment
3	40 – 60	Minor rehabilitation
2	25- 40	Major rehabilitation
1	< 25	Reconstruction

7.3 Deflection Criteria

7.3.1 Relationship Between Tolerable Deflection and Cumulative Traffic

Deflection tests are carried out using the FWD, HWD (heavy-weight deflectometer), Benkelman beam, and Light Weight Deflectometer, see RDM 5.1. Generally, the lower the stress in a material, the more stress repetitions are required for failure to occur. For a given form of flexible pavement, the magnitude of the surface deflection is an indicator of the strains in the pavement layers and the subgrade. Therefore, a given pavement structure has a limiting deflection which is a function of the number of load applications As yet the theoretical determination of the relationship between deflection and future traffic-carrying capacity is still uncertain. This relationship must therefore be established empirically. Its graphic representation is called a “deflection criterion curve”.

It is emphasised that there is a different relationship between allowable (or “critical”) deflection and cumulative traffic for each type of flexible pavement and each type of subgrade. Details are given in the Performance Design Method also known as the Deflection Reduction Method covered in Section 9.8.

7.3.2 Life Phases of Flexible Pavement Deflection Changes

The deflection of a flexible pavement does not remain constant with time. The deflection life of a well-designed flexible pavement can be divided into the following four distinctive phases, Table 7.4.

Table 7.4 Life Phases of Flexible Pavement Deflection

Item No.	Phases	Description
1	**Consolidation Phase**	Pavement layers and the subgrade undergo consolidation due to the action of the wheel loads. This causes a slight decrease in the surface deflection. This phase is relatively short and the magnitude of consolidation depends on the compaction received by the various layers during construction. Stabilising the subgrade moisture content may also produce consolidation. Ruts may appear in the wheel paths. This phase will be almost unnoticeable if the various layers have received sufficient compaction during construction
2	**Elastic Phase**	During this phase, a genuine elastic performance occurs. Each wheel load produces a deflection, which recovers completely after the passage. The deflection does not change during the elastic phase and is termed “early life deflection”. This constant early life deflection constitutes an important characteristic. It should be measured after the initial changes (consolidation) occur, about 6 months after the road opening.
3	**Plastic Phase (fatigue)**	Due of traffic and climate, the pavement deteriorates gradually by attrition in the upper layers, compression in the subgrade, loss of cohesion in the treated materials and ageing of the bitumen. Each wheel load produces a small, non-recoverable deformation (“residual deflection”). During this plastic phase, the deflection increases slowly. Ideally, the pavement should be strengthened towards the end of this phase before extensive deterioration occurs.
4	**Failure**	If the pavement is not strengthened in time, it deteriorates rapidly by deformation and disruption. In the absence of heavy repairs and under adverse climatic conditions, holes form which spread and destroy large parts of pavement. There is a sharp increase in deflection during this phase.

Notes:

i. *In the case of a poorly designed flexible pavement, there can be no elastic phase. If so plastic deterioration and fatigue will affect the pavement immediately after it is opened to traffic.*

ii. *In the case of a rigid pavement, there is no plastic phase. The pavement may break up suddenly by fracture without showing previous signs of deterioration. In particular, there is no gradual increase in the deflection before failure. Consequently, the deflection cannot be used as an indicator of the structural adequacy of rigid pavements.*

iii. *Some maintenance operations may influence performance, particularly resealing and drainage improvement. It is, therefore, essential to know and the extent of all the principal maintenance operations.*

7.3.3 The Use of Deflection for Performance Prediction and Residual Life Estimation of Flexible Pavements

For a given type of flexible pavement, it is necessary to know both the relationship between allowable (critical) deflection and cumulative traffic and the variation of the actual deflection against cumulative traffic to predict the future performance of a road.

Figure 7.1 illustrates a notional deflection criterion curve, obtained by drawing a line between the plots of the deflections of pavements in poor and acceptable conditions. In the same figure, a typical curve representing the variation of the actual deflection against cumulative traffic is shown.

Knowledge of critical deflection and deflection history enables the pavement life expectancy to be estimated. As indicated, a pavement having a deflection D1 after carrying N1 standard axles has a residual life equal to (N2-NI) standard axles, N2 corresponding to the critical deflection D2.

Conversely, if it is desired that the pavement is considered to have carried N2 standard axles during its design life, its early life deflection should not exceed Do.

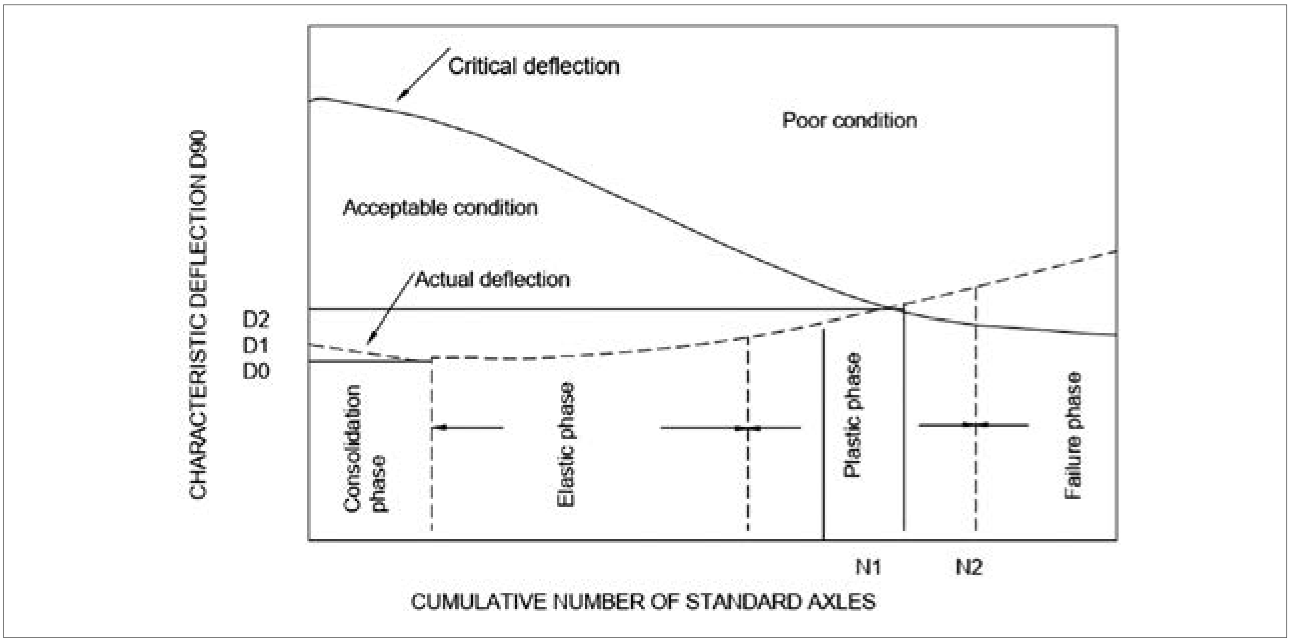
This may be used to check, shortly after construction, if the pavement is adequate.

Thus, to be able to plan to strengthen a given flexible pavement it is necessary to know:

1. Characteristics of the pavement structure.
2. Characteristics of the subgrade.
3. Relationship between critical deflection and cumulative traffic.
4. Probable variation of the deflection against traffic.
5. Cumulative traffic carried until the time of deflection measurement.

One deflection survey alone is insufficient to evaluate structural adequacy or to predict future performance or residual life.

Figure 7.1 Illustration of Deflection Criteria Curve



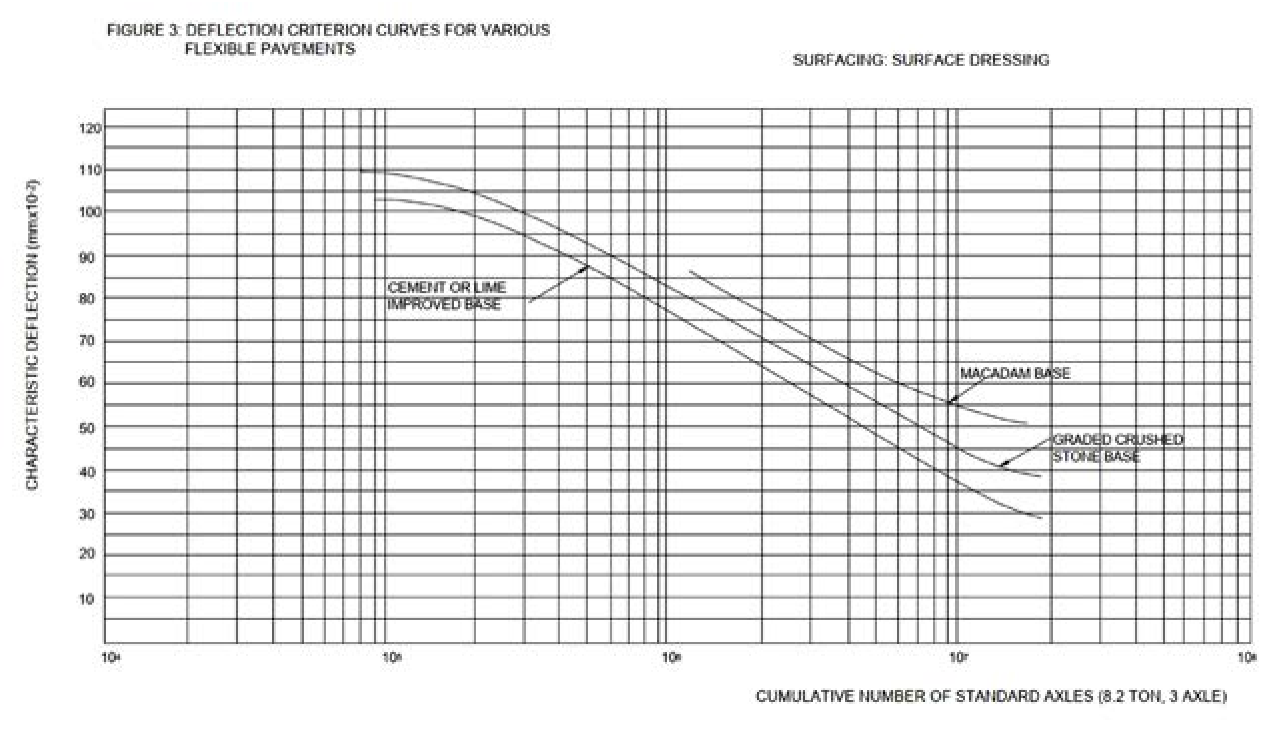
7.3.4 Relations of Tolerable Deflection and Cumulative Traffic Obtained in Kenya

It has been possible to establish the relationship between allowable deflection and cumulative traffic for 3 types of flexible pavements, namely:

1. **Surfacing:** Surface dressing or thin asphaltic concrete.
   Surfacing: (25 - 30 mm cracked).
   Base: 125 - 175 mm graded crushed stone.
   Subbase: 100 - 200 mm flexible material.
2. **Surfacing:** Surface dressing
   Base: 125 - 175 mm cement or lime improved material.
   Subbase: 75 - 150 mm flexible material.
3. **Surfacing**: Surface dressing.
   Base: 100 - 200 mm water-bound macadam.
   Subbase: 100 - 250 mm natural gravel.

The corresponding curves are shown in Figure 7.2.

Figure 7.2 Characteristic/critical Deflection for Flexible Pavements in Kenya



7.4 Product RD Criteria

The calculations for ‘R’ and ‘D’ are provided in RDM 5.1.

Theory indicates that, for an ideally uniform section of pavement, the product 'RD' ('R': radius of curvature, 'D': deflection) depends only on the thickness 'h' of the pavement’s upper layers and the ratio of the moduli of elasticity E1/E2, if the pavement structure is considered as a two-layer system.

Since no section of pavement is perfectly uniform considerable scatter of the product RD is often observed, nevertheless it is thought that, for a given type of pavement and subgrade, there is a critical value of the product RD below which the structural condition is inadequate.

8 Techniques and Materials for Road Strengthening

8.1 General

Overlaying a pavement is the most often adopted method of increasing its traffic-carrying capacity, however, it is not the only one. Due consideration should always be given to other means of strengthening, particularly, the restoration and improvement of the road drainage system.

8.2 Restoration and Improvement of the Drainage System

If pavement deterioration can be attributed to poor drainage, the first strengthening operation to undertake is to correct any faults in the drainage system

Drainage deficiencies may result from the poor design of the system or changes in the road environment, particularly in the run-off conditions. A complete assessment of drainage should be made to determine the required corrections.

8.2.1 External and Subsurface Drainage

An assessment of the adequacy and the state of the surface drainage system (topography, ditches, drains, pipes, culverts etc.) will enable the necessary corrective measures to be decided, Table 8.1.

Table 8.1 Significance of Surface and Ground Improvements Required

Item No.	Drainage system	Challenges	Remedies
1	**Surface Drainage**	**a.** Lack of maintenance **b.** Inadequate design **c.** Extreme flooding	**a.** Clear the ditches and drains of vegetation, debris and sediment and repair any erosion damage. **b.** Widen and deepen the side ditches and outfalls. **c.** Construct additional catchwater or discharge drains. **d.** Line ditches the sides of which erosion causes collapse. **e.** Replace pipes of insufficient section. **f.** Place additional pipes.
2	**Ground/ Subsurface Drainage**	**a.** In cuttings, the road pavement and subgrade may be adversely affected by groundwater (water table or springs); usually, this problem affects only short lengths of road. **b.** In low-lying or poorly drained flat areas, where a water table near formation saturates the subgrade by capillary flow.	**a.** Subsurface or blanket drains should then be placed to cut off seepage lines or to lower the water table **b.** If a short road length is involved, the best solution is to raise the formation level and reconstruct the pavement. **c.** If a longer stretch of the road is affected, alternative routes should be considered if a less costly long-term solution cannot be found.
3	**Internal Pavement Drainage**	Pavements with a permeable base (generally of stone) and having impermeable shoulders and subbase ('trench' type of cross-section). Water entering the pavement, through cracks on the surface or defective edge seals, cannot be drained away and accumulates in the base and/ or subbase (Figure 8.1) causing the rapid failure of the whole structure.	**a.** Provide a continuous drainage layer. **b.** Removing the existing impervious shoulders **c.** Place a continuous drainage layer at the level of the underside of the GCS **d.** Reinstate the upper shoulder with cohesive and impermeable gravel **e.** Resealing the surface, including edge seals. **f.** Approved filter materials (>75mm thick) or geotextile should be used and daylighted.

Figure 8.1 Example of Waterlogged Pavement Layers

8.2.2 Carrying Out Drainage Works in Advance of the Overlay and Re-Evaluate the Structural Conditions

Rectification of any drainage deficiencies should be carried out at least 6 months and more appropriately, 12 months ahead of the strengthening works so that the saturated layers can dry out and consolidate. This way, at least one rainy season will have put the improved drainage works to the test before the works.

It is necessary to re-evaluate the structural condition of the pavement after drying out and consolidation. Indeed, drainage improvements may increase the pavement strength and enable a reduction in the proposed overlay thickness.

8.3 Preparation of Existing Pavement for Overlays

The overlay thickness is designed to provide the additional strength required for the bulk of existing pavement (normally about 90%), but not for local weak spots. If the overlay design was based on the weakest spot, the section would be overdesigned and unjustifiably too expensive. In practice, these weaker areas must be identified and rectified before overlying. See Section 9.7.5 on rationalising overlay thickness. evaluating road pavement defects:

8.3.1 Local Repairs

All extensively deteriorated areas, i.e., potholes, depressions and areas exhibiting cracking >3mm and spalling (crazing with loose fragments) should be properly patched.

Using patching material with stiffness comparable to that in the existing pavement is recommended, otherwise, differential settlements and cracks may occur at the joints.

If the pavement edges are damaged, special care must be taken to repair them with strong material to ensure an adequate edge restraint. The most usual patching materials are cold-mix asphalt or pre-mix and cement-improved gravel.

All loose materials resulting from surface deterioration (fine crazing, ravelling, stripping, etc.) should be removed and all open cracks sealed.

8.3.2 Levelling

When the existing surface is extensively deformed, use regulating courses or levelling wedges to restore proper line and cross-section, thus enabling smooth laying and uniform compaction of the subsequent overlay.

A thorough geometric survey of the existing surface should be carried out to determine the make-up quantities required.

Levelling wedges are patches of material used to fill up sags and depressions or to reduce an excessive crown. The most practical levelling method will depend on the type, amplitude, and distribution of surface irregularities and the nature and thickness of the overlay.

When the overlay thickness required is large, it is current practice to place the overlay in several layers with the first one acting as a regulating course.

Where only a small overlay thickness is required, the pavement is generally still in fairly good condition and therefore little deformed. However, when the pavement's structural condition warrants only a comparatively thin overlay whilst the existing surface exhibits pronounced irregularities, it will be necessary to carry out specific levelling operations to correct the existing surface and permit proper placing of the thin overlay.

The most suitable material for such levelling wedges is cold asphalt. The binder may be either a bitumen emulsion or a medium-curing cut-back. A tack coat is required. The mixture should have a maximum particle size that permits feathering at high spots in the pavement. The most appropriate way of placing such levelling wedges is by motor-grader, paver, or other methods depending on the areas to be laid and the plant available.

Ruts can also be filled with a cold mix or hot mix AC before applying a surface dressing or thin overlay. Ruts that can be filled should not show signs of shoving or any lateral movement of pavement materials.

8.3.3 Cleaning and Tack-Coat

Immediately before the overlay application, the surface must be thoroughly swept and all loose and foreign material shall be removed.

If bituminous overlay (e.g., asphaltic concrete or gap-graded asphalt or sand asphalt) overlay is to be placed, it is generally necessary to spray a tack-coat to ensure uniform and complete adherence of the overlay. The material for the tack coat should be hot bitumen.

8.4 Overlays

Overlays can be Non-structural or Structural (discussed in the next sections). They are usually asphalt but concrete overlays can also be used, depending on factors such as the type and condition of the existing pavement, the required life and the budget available.

An overlay is just one of the rehabilitation options that should be considered. Other options include shallow and deep inlays (asphalt or concrete) to a single lane or the whole carriageway, or a surface dressing/Otta seal/etc to seal cracks and restore skid resistance.

An overlay is unlikely to fix drainage-related structural failures or issues with weak granular layers or subgrades, but if these areas are localised then local reconstruction could be carried out before an overlay. Some types of overlays, e.g. CRCP (concrete), are good at load spreading so will be more tolerant of poor/variable subgrades.

Pavement construction details, condition survey information and required pavement life will need to be examined to determine the required overlay thickness. It may be that raising the road level with the associated works (such as overhead bridge height clearances, replacing safety fencing and raising the height of side roads, etc) means that a thick overlay is not viable.

8.4.1 Non-Structural Overlays

NON-STRUCTURAL overlays (generally 30-50mm thick) are carried out when the existing pavement is structurally sound, but the road surface needs renewing. The principal reasons for overlaying otherwise adequate pavements are as follows:

1. Excessive surface roughness, due to ravelling, peeling, scaling, spalling or patching.
2. Surface deformation, due to rutting, shoving or corrugating of the existing surfacing, settlement, etc.
3. Loss of skid resistance due to worn surface.

The new overlay is usually placed directly onto the underlying pavement, without an interlayer (separation layer). Any distresses or areas of weakness/movement in the underlying pavement (e.g. movement at a joint/crack) must be accommodated in the overlay to avoid damage.

Non-structural overlays are generally bituminous premixes such as asphaltic concrete, gap-graded asphalt or sand-asphalt (maximum particle size will depend on the thickness required), but a thin concrete layer can also be used (see UTRCP in Section 10.6).

Apart from their specific purpose of restoring the riding quality they also waterproof the surface.

8.4.2 Structural Overlays

Structural overlays (generally 100-250mm thick) are required when the pavement is approaching the end of its life. Instead of reconstruction, rehabilitation in the form of a thick overlay is used to extend the pavement's design life.

Structural overlays for existing asphalt and concrete pavements can be made of asphalt or concrete. Each type of overlay will have a different behaviour, with its pros and cons.

If a concrete overlay is to be used, then this can either be a Bonded Concrete Overlay (BCO) where the concrete is laid directly onto the existing pavement or an Unbonded Concrete Overlay (UBOL) where an interlayer (also called a separation layer), usually asphalt, is placed between the existing pavement and the new concrete overlay (this is discussed further in Section 8.4.5.2).

8.4.3 Flexible Overlay Materials

Experience shows that flexible overlay materials are suitable for light and medium traffic; classes TC1-TC3-TC10, a cumulative traffic loading of up to 10 million CESA.

The flexible overlay materials consist of a flexible base course covered with either surface dressing or not more than 50 mm of flexible asphaltic concrete. Three key materials suitable for the base course are cement or lime-improved gravel, bitumen stabilised based and graded crushed stone (GCS). The standards for flexible overlayers are given in Table 8.2. Additional information is provided in RDM 3.3 and RDM 3.4.

Table 8.2 Standards for Overlay Materials

Item No.	Overlay Materials/ Layers	Traffic Loading Limits	Design Considerations
1	**Flexible pavement layers**	**Light traffic:** TC3 Labour-based surfaced roads – TC0.1 **Labour based block paving:** TC1	Unpaved and paved. Low-volume sealed roads **Surfacing:** SSD or DSD or cape seal, penetration macadam, Otta seal, cold mix AC, etc. are applicable. **Bases:** Natural, HIG and GCS. Low plasticity bases (G60, G80, etc) preferred - PM<100. **Maintenance:** Fog spray at 4 years and reseal at 7 years.
		Medium traffic: TC3 -TC10	**Surfacing:** DSD, TSD, thin AC, penetration macadam, Cape Seal, Otta Seals **Bases:** natural gravels, GCS, HIG, mechanically stabilised bases, etc. **Maintenance:** Fog spray at 4 years and reseal at 7 years.
		High traffic: TC10 – TC30	**Surfacing:** DSD, TSD, penetration macadam, cold mix AC, thin AC. DBM+SSD/DSD, DBM+AC **Bases:** natural coarse bases (incl. Macadam and Telford), HIG, GCS, HBS, etc. **Maintenance:** Fog spray at 4 years and reseal at 7 years.
2	**Hydraulically Improved Materials, HIG** (CBR: min 160 or I200kN/m^2)	**Fine materials:** TC3 **Low plasticity paedogenic materials:** TC10 **Low plasticity quartzite**: TC10	**Surfacing:** For TC3 - DSD, TSD Penetration macadam, Cape seal, hot sand asphalt, For TC10 – DSD, TSD, Penetration macadam, Cape seal, hot sand asphalt, ACI-50+SSD, ACII-50+SSD **Maintenance:** Fog spray at 4 years and reseal at 7 years.
		Coarse material (nominal max -37.5mm): TC30	**Surfacing:** Thin DSD, TSD, ACII-50 mm, ACII-50+SSD, (flexible) ACI-50+SSD or gap-graded asphalt or sand-asphalt. **Maintenance:** Fog spray at 5 years and reseal at 10 years.
		Very coarse e.g., macadam (nominal max 70mm) or Telford (nominal max. – 150 mm): TC80	**Surfacing:** DBM80+DSD/TSD Medium graded AC50 **Maintenance:** Fog spray at 5 years and reseal at 10 years.

Table 8.2 Standards for Overlay Materials *(continued...)*

Item No.	Overlay Materials/ Layers	Traffic Loading Limits	Design Considerations
3	Graded Crushed Stone	**Class D:** TC3	**Surfacing**: For TC3 - DSD, TSD Penetration macadam, Cape seal, hot sand asphalt. **Maintenance:** Fog spray at 4 years and reseal at 7 years.
		Class C: TC10	**Surfacing:** For TC10 – DSD, TSD, ACII-50Penetration macadam, Cape seal, ACI-50+SSD **Maintenance:** Fog spray at 4 years and reseal at 7 years.
		Class B: TC30	**Surfacing:** DSD, TSD, Penetration macadam Thin ACII-50, ACI-50 or gap-graded asphalt or sand-asphalt. **Maintenance:** Fog spray at 5 years and reseal at 10 years.
		Class A: TC80 Class A with cement treatment: TC80	**Surfacing:** DSD, TSD, Penetration macadam, Thin ACII-50, ACI+SSD or gap-graded asphalt or sand-asphalt. Medium graded AC50 **Maintenance:** Fog spray at 5 years and reseal at 10 years.
4	Bitumen-stabilised materials (BSM)	**Fine materials** (incl. sand) **BSM1 (5-6%):** TC1 Foamed bitumen Emulsion treated base	**Surfacing:** DSD, TSD Penetration macadam, Cape seal, hot sand asphalt, ACII-50 or gap-graded asphalt or sand-asphalt. Medium graded ACI-50+SSD **Maintenance:** Fog spray at 4 years and reseal at 7 years.
		Medium graded BSM3 (5-6%): TC3 Foamed bitumen	**Surfacing:** DSD, TSD Penetration macadam, Cape seal, hot sand asphalt, ACII, etc. **Maintenance:** Fog spray at 4 years and reseal at 7 years.
		Coarse graded BSM6 (5-6%): TC10 -TC30 Foamed bitumen or emulsion-based	**Surfacing:** DSD, TSD Penetration macadam, Cape seal, hot sand asphalt, ACII-50 or gap-graded or sand-asphalt. Medium graded ACI-50+SSD **Maintenance:** Fog spray at 4 years and reseal at 7 years.
		ETB (5-6%): TC30 Emulsion-treated base or formed bitumen base	**Surfacing:** For DSD, TSD Penetration macadam, Cape seal, hot sand asphalt, DBM+SSD, AC11-50, ACI-50+SSD, etc. **Maintenance:** Fog spray at 4 years and reseal at 7 years.
5	Blended or mechanically stabilised gravels (gravel + GCS	**Well graded:** TC10	**Surfacing:** DSD, TSD Penetration macadam, Cape seal, hot sand asphalt. **Maintenance:** Fog spray at 4 years and reseal at 7 years.
		Well graded: TC30	**Surfacing:** DSD, TSD Penetration macadam, Cape seal, hot sand asphalt, DBM+SSD, ACII-50, ACI-50+SSD, gap-graded asphalt, hot sand asphalt, medium graded AC50 **Maintenance:** Fog spray at 4 years and reseal at 7 years.

Flexible Pavements:

The materials shall comply with Table 6.4. Such materials can be employed under double surface dressing for light traffic up to 3 million standard axles, classes 1 MCESA-3 MCESA.

1. Medium traffic (3 MCESA-10 MCESA) requires:
    a. Either 50 mm flexible asphaltic concrete as surfacing, or
    b. A fairly good resistance to attrition if surface dressing is applied.

    These materials are unsuitable for traffic over 10 million standard axles, owing to their insufficient strength and resistance to attrition.

    The surfacing and base-course requirements for this type of overlay can be summarised as follows:

2. Traffic Classes 1 MCESA-3 MCESA require:
    a. **Surfacing:** Double surface dressing
    b. **Base:** Cement or lime-improved material, Table 6.4, (CBR: min 160)

3. Traffic Class 10 MCESA

    Alternative (A)

    a. **Surfacing:** 50 mm asphaltic concrete Type II (flexible) or gap-graded asphalt or sand-asphalt. However, DSD, TSD and Cape Seal can also be used.
    b. **Base:** Cement or lime-improved material, Table 6.4, (CBR: min 160).

    Alternative (B)

    a. **Surfacing;** Triple surface dressing.
    b. **Base Cement or lime improved material** (LAA: max. 50, ACV: max. 35, UCS: min l200kN/m^2).

This type of overlay is relatively cheap, however in some regions, particularly the volcanic areas, materials suitable for treatment are scarce.

Care should be taken to ensure the overlay material is not too stiff to avoid overstressing and cracking. If the treated material has appreciable rigidity, the required overlay thickness is determined similarly to cement-stabilised gravel.

Treated material should not be laid in layers of compacted thickness less than 125 mm and treated clayey sand in layers less than 100mm.

1. **Graded Crushed Stone**

    The graded crushed stone should comply with the requirements given in Table 6.5. Graded crushed stone can be used under double surface dressing for light traffic up to 3 million standard axles (1 MCESA-3 MCESA). Whilst medium traffic (3 MCESA to 10 MCESA) requires:

    a. Either 50 mm flexible asphaltic concrete as surfacing or
    b. Sufficient angularity and hardness of the stone if the surface dressing is applied, DSD, TSD, Cape seal.

    The surfacing and base course requirements for this type of overlay are summarised as follows:

2. Traffic Classes 1 MCESA-3 MCESA
    a. **Surfacing:** Double surface dressing
    b. **Base:** Graded crushed stone Class C

1
2
3
4
5
6
7
8
Techniques and Materials for Road Strengthening

3. Traffic Class 30 MCESA

Alternative (A)

a. **Surfacing:** 50 mm asphaltic concrete Type II (Flexible)
b. Gap-graded asphalt or sand-asphalt, or
c. **Base:** Graded crushed stone Class B

Alternative (B)

a. **Surfacing:** Triple surface dressing
b. **Base:** Entirely crushed G.C.S. Class B

Graded crushed stone is considered where gravel suitable for the base is unavailable. Graded Crushed Stone prevents cracks from reflecting through to the surface of the overlaid pavement.

The minimum compacted thickness that can be practically placed is 125 mm for 0/40mm granularity and 100 mm, for 0/30 mm granularity.

Note: *If suitable stone for chippings cannot be found, a thin layer (25 mm+) of asphalt concrete (Type II), gap-graded or medium-graded asphalt or sand-asphalt may be placed instead of surface dressing.*

4. **Other Flexible Overlay Materials**

Consideration can be given to other flexible materials, such as:

a. Low-plasticity gravel treated in-situ with bitumen. These include bitumen-stabilised materials and BSM. The bitumen could be emulsion (emulsion treated based, ETB) or materials stabilised with foamed bitumen. Usually after stabilisation, the residual bitumen in the mix should be 5-6% by mass.
b. Graded crushed stone treated in-situ with a low content (2%) of bitumen emulsion, though in some jurisdictions cement treatment of GCS is considered unnecessary (e.g., South Africa standards), or
c. A mixture of gravel and graded crushed stone possibly treated with lime or a small amount of cement. Blending is commonly used where materials are scarce. Such blending should be carried out in the laboratory and the properties of the blend should be tested for conformity with the specifications and standards.

8.4.4 Bound Overlay Materials

Bound materials whose failure criterion is cracking rather than deformation. This includes thick asphalt concrete type 1 and type 2, dense bituminous mixes (DBM), dense emulsion mixes (DEM), sand asphalt, hydraulically bound materials (cement and lime stabilised materials), Hydraulically bound stone, etc. The application of bound materials for overlays are given in Table 8.3.

Table 8.3 Standards for Bound Overlay Materials

Item No.	Overlay materials/ layers	Traffic loading limits	Conditionality
1	**Asphaltic Concrete** (Continuously Graded Asphalt)	**ACI:** TC80+	**Surfacing:** None in temperate to subtropical climate SSD, DSD, TSD in hot climate – SSD is preferred for economic reasons. **Maintenance:** Fog spray at 4 years and reseal at 7 years.
		ACII: TC10	**Surfacing:** None in temperate to subtropical climate SSD, DSD, TSD in hot climate – SSD is preferred for economic reasons. **Maintenance:** Fog spray at 4 years and reseal at 7 years.
2	**Gap-Graded** ('Hot-Rolled') **Asphalt**	**Fine materials:** (nominal max size 10mm) **AC50:** TC3	**Surfacing:** DSD, TSD Penetration macadam, Cape seal, hot sand asphalt. **Maintenance:** Fog spray at 4 years and reseal at 7 years.
		Coarse material (nominal max -12-16mm): TC10	**Surfacing:** None in temperate to subtropical climate SSD, DSD, TSD in hot climate – SSD is preferred for economic reasons. **Maintenance:** Fog spray at 4 years and reseal at 7 years.
		Medium graded AC: TC50	**Surfacing:** None in temperate and subtropical climate SSD, DSD, TSD in hot climate – SSD is preferred for economic reasons. Not required for medium-graded AC. **Maintenance:** Fog spray at 4 years and reseal at 7 years.
3	**Sand Asphalt**	**Hot sand asphalt, 50mm:** TC3	**Surfacing:** None in temperate and tropical climate. **Maintenance:** Fog spray at 4 years and reseal at 7 years.
		Hot sand asphalt, 50mm: TC10	**Surfacing:** SSD, DSD and TSD, Cape seal for improved skid resistance and durability. **Maintenance:** Fog spray at 4 years and reseal at 7 years.
4	**Dense Bituminous Macadam** (DBM)	**DBM (0/40mm):** TC30 **Minimum:** TC10 for economic reasons	**Surfacing:** ACI-50, DSD, TSD, Cape seal, and hot sand asphalt. **Maintenance:** Fog spray at 4 years and reseal at 7 years.
		DBM (0/30mm): TC80 **Minimum:** TC10 for economic reasons	**Surfacing:** ACI-50, DSD, TSD, Cape seal, and hot sand asphalt. **Maintenance:** Fog spray at 4 years and reseal at 7 years.
5	**Dense Emulsion Macadam** (DEM)	**DEM (0/40mm):** TC30 **Minimum:** TC10 for economic reasons	**Surfacing:** ACI-50, DSD, TSD, Cape seal, and hot sand asphalt. **Maintenance:** Fog spray at 4 years and reseal at 7 years
		DEM (0/30mm): TC80 **Minimum:** TC10 for economic reasons	**Surfacing:** AC type 1, DSD, TSD, Cape seal, and hot sand asphalt. **Maintenance:** Fog spray at 4 years and reseal at 7 years.

1. Asphaltic Concrete (Continuously Graded Asphalt)

This is the most common overlay material, usually asphaltic Concrete (Type I) and is used to resist rutting and high stresses. The more commonly used rut resistance asphalt materials are Superpave and EME (see RDM 3 for further information on Superpave and EME).

However, this material cannot be laid in thin layers on a deformable support because it would be overstressed. Appreciable thicknesses are hence necessary if it is to be used for overlays on flexible pavement.

Strengthen deformable pavements with more flexible materials such as asphaltic concrete (Type II), gap-graded, medium-graded or sand asphalt.

The material specifications, traffic limitations and construction procedures for asphaltic concrete are summarised in Table 6.4.

Notes:

a. *Due to the rapid hardening of bitumen in Kenya, using bitumen harder than 60/70 penetration grade is not recommended unless a seal is put on top.*

b. *Asphaltic concrete (Type II) is suitable only for light and medium traffic (up to 10 million standard axles).*

c. Minimum practical thickness laid by the current types of pavers is 25 mm (provided the maximum aggregate size does not exceed 10 mm).

2. Gap-Graded ('Hot-Rolled') Asphalt

Table 6.4 summarises the materials requirements, traffic and construction limitations applicable to gap-graded mixes suitable for thin-wearing courses (25-50 mm) and light to medium traffic (1 MCESA-3 MCESA- 10 MCESA). Such mixes are expected to be flexible, fatigue-resistant and durable, owing to the good distribution of the void structure and the rounded shape of most fine aggregate.

Gap-graded asphalt may be suitable for heavier traffic or thicker layers, subject to its compliance with severe specifications; in particular, better resistance to rutting is required. However, due to the rapid ageing of bitumen, a compromise between resistance to rutting and resistance to fatigue may be difficult to achieve. A surface dressing can be placed on the gap-graded AC to prevent rapid ageing and oxidation.

1. Sand Asphalt

The use of sand asphalt is a viable solution where suitable stone cannot be sourced economically. The materials' requirements, traffic and usage limitations are given Table 6.4.

As its resistance to rutting is low, sand asphalt is only suitable for thin-wearing courses (<50 mm) and light to medium traffic (1 MCESA-3 MCESA-10 MCESA).

2. Dense Bituminous Macadam (DBM)

The materials requirements, traffic limitations and construction procedures are summarised in Table 6.4. The following points should be noted:

- **a.** Because of the rapid ageing of bitumen observed in Kenya, the use of 40/50-grade bitumen is not recommended unless the layer is sealed with SSD or DSD.
- **b.** Suitable ranges of compacted thickness to be laid are as follows:
  - **i.** 60 -100 mm for 0/30 mm granularity
  - **ii.** 75- 125 mm for 0/40 mm granularity
- **c.** Traffic limitations require 0/30 mm DBM for traffic class ≥ 80 MCESA, and 0/40 mm. DBM for traffic classes 10 MCESA-30 MCESA. However, depending on the thickness required 0/30mm DBM may be used for traffic classes 3 MCESA and 10 MCESA.
- **d.** To provide for imperviousness and good riding quality it is usual to place a wearing course of 50 mm thick asphaltic concrete on top of the dense bituminous macadam. It is also possible to cover the dense bituminous macadam with surface dressing. Use a denser DBM grading and a slightly higher bitumen content and pay special attention to the evenness of the DBM surface.

e. Dense bituminous macadam is adequate for all traffic but is only economically viable for heavy traffic (≥30 MCESA).

f. Dense bituminous macadam must be protected immediately with a wearing course to prevent the percolation of rainwater into it.

g. Dense bituminous macadam may be trafficked immediately after compaction; this is an appreciable advantage over the cement-treated material.

3. Dense Emulsion Macadam (DEM)

Dense emulsion macadam is a cold mixed, cold laid, plant-mix of well-graded aggregate and bituminous emulsion.

The materials' requirements, traffic limitations and construction procedures are summarised in Table 6.4.

Dense emulsion macadam has several advantages:

a. It is economical: cold-mixing equipment is comparatively simple and cheap. Large production outputs can be maintained. No heating is required.

b. It is easy to use: no temperature control is necessary. Moreover, the mix can be laid by a grader and therefore lends itself remarkably well to regulating and levelling operations.

c. Dense emulsion macadam can be placed in comparatively thin layers 75 mm for 0/30 mm DEM and 100 mm for 0/40 mm DEM.

d. When properly formulated and placed, its strength is marginally less than that of dense bitumen macadam. On the other hand, as the binder is used cold, it is not oxidised during mixing and laying and remains tacky and ductile. This gives a good fatigue resistance dense emulsion macadam.

e. The following points are also stressed:

f. The granularity 0/30 mm is required for traffic classes ≥ 30 MCESA (TC30 and TC80) . The 0/40 mm DEM can be employed for ≤ 10 MCESA, if the thickness required exceeds 100 mm.

g. To adequately protect the base against excessive stresses and attrition, and to obtain sufficient imperviousness and satisfactory riding quality, asphaltic concrete surfacing is required for heavier traffic (30 MCESA-80 MCESA):

    i. 75 mm for traffic class ≤ 80 MCESA.

    ii. 50 mm for traffic class ≤ 30MCESA.

h. Surface dressing is adequate for medium traffic (3 MCESA-10 MCESA) but can be used for heavy traffic (80 MCESA)

i. Dense emulsion macadam is suitable for all traffic but is economically justified only for heavy and medium traffic (10 MCESA-30 MCESA-80 MCESA).

j. Heavy compaction is required for dense emulsion macadam. The adjustment of the moisture content is of prime importance.

k. Dense emulsion macadam must not be placed in layers of compacted thickness exceeding 150 mm, so that water can evaporate. It shall be allowed to “cure” before any covering is applied.

Dense emulsion macadam can be trafficked immediately after compaction (the action of traffic is beneficial as it provides additional compaction).

1
2
3
4
5
6
7
8
Techniques and Materials for Road Strengthening

4. Cement Stabilised Gravel

The materials requirements are summarised in Table 6.4.

The following points are emphasised :

- **a.** To ensure the uniformity of the mix, it is advisable to use a stationary mixing plant. In this respect, it should be noted that only low-plasticity materials (plasticity modulus not exceeding 700) can be properly mixed in stationary plant.
- **b.** Attention is drawn to the time limitations imposed by the rapid setting of the cement. 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.
- **c.** No vehicle should be permitted on cement-stabilised gravel for at least 7 days.
- **d.** For heavy and medium traffic asphaltic concrete surfacing is necessary to avoid excessive shear stresses and attrition in the stabilised gravel. The required thicknesses of asphaltic concrete (Type I) are:
  - **i.** 50 mm for traffic Class ≤ 10 MCESA.
  - **ii.** 75 mm for traffic Class ≤ 30 MCESA.
  - **iii.** 100 mm for traffic Class ≤ 80 MCESA.
- **e.** Surface dressing is adequate for light traffic.
- **f.** This type of overlay is comparatively cheap, considering the other structures required for heavy traffic. It compares advantageously with hydraulically bound stone, in particular, because of its more favourable ratio of tensile strength to elastic modulus.

Unfortunately, suitable gravels are scarce in many parts of the country, especially in the volcanic regions.

5. Hydraulically Bound Stone:

The material requirements are summarised in Table 6.4. The following points should be noted:

- **a.** Hydraulically bound stone as an overlay material has several advantages; its characteristics are independent of the temperature and a high modulus of elasticity is obtained.
- **b.** Due to its rigidity, hydraulically bound stone must be placed in thick layers (minimum 150 mm).
- **c.** Widely spaced cracks, due to shrinkage and thermal changes, are inevitable. Care must be taken to ensure that the pavement layers are properly drained. A continuous drainage layer through the shoulders is needed.

Eliminating reflection cracking would require considerable bituminous wearing course thickness, which is regarded as economically unreasonable and technically unnecessary. Despite their unpleasant psychological effect, reflection cracks do not affect the riding quality and have no effect structurally, if the overlay is adequately designed and constructed.

An asphaltic surfacing of 75 and 50 mm has consequently been adopted for traffic Classes 80 MCESA and 30 MCESA respectively, mainly for imperviousness and good riding quality. Surface dressing has been considered suitable for traffic Class ≤ 30 MCESA.

1. Attention is drawn to the time limitations due to the rapid setting of cement. Compaction should be completed not less than two hours after starting of mixing and protection against evaporation be required not less than 4 hours after completion of compaction.
2. No vehicles should be permitted on lean concrete for at least the first 7 days.

8.4.5 Preventing Reflection Cracking

Cracks are caused by differential vertical and horizontal movements above the crack tip, referred to as crack activity. This is caused by thermal stresses or traffic loading or both. Crack activity causes stress concentration on the overlay above the crack causing cracks to develop in the overlay with the same pattern as the underlaying layer.

When overlaying a cracked pavement, special care must be taken to prevent any existing cracks from reflecting through to the overlay surface. This point is of prime importance in the case of an existing rigid or semi-rigid pavement exhibiting large and open cracks. Three methods can be employed to eliminate reflection cracking; these are as follows:

8.4.5.1 Crack, Seat and Overlay

The Crack, Seat and Overlay (CSO) technique (also called Crack and Seat or C&S) has been well used in the UK and USA to extend the life of a failed concrete or composite pavement. It is usually used for unreinforced JUC but another version called Saw-cut, Crack, Seat and Overlay (SCCSO) can be used on reinforced concrete. The CSO technique involves:

1. Breaking the existing concrete (or hydraulically bound material) into approximately 1m length blocks using a specialist Guillotine Breaker and then seating the blocks with a Pneumatic Tyred Roller (PTR). Field trials may be required to verify the practicality and effectiveness of such a method since each case will require individual study and comparative cost analysis.
2. Placing a sufficient thickness of overlay (usually 180mm asphalt). This method is generally costly as the overlay thickness required is considerable:
   a. Bituminous mixes: – absolute minimum 120 mm.
   b. Cement-stabilised gravel: – absolute minimum 180 mm.

8.4.5.2 Crack Prevention Using Intermediate Layer

1. Placing an intermediate layer of unbound granular material ('cushion course'). This method is very effective and is therefore recommended.
2. GCS - The most suitable cushion material is graded crushed stone.
   a. For high volumes roads, a minimum thickness of 100 mm should be applied.
   b. For low-volume roads, a minimum thickness of 50 mm is sufficient.
3. Natural gravel – material should be granular and non-cohesive

8.4.5.3 Use of Geogrids, Geofabric and Geosynthetics for Asphalt Reinforcement

Geogrids have the effect of reinforcing against the reflection of cracking from cracked asphalt and hydraulically bound layers into uncracked upper layers or overlays. Generally, reinforcement can be considered in:

1. Asphalt overlays over cracked pavements
2. Interface between cement-stabilised layers and surfacing
3. Asphalt surfacing at high-stress areas such as climbing lanes, intersections and freight terminals
4. Airfield runways and taxiways
5. Strengthening layers made of natural materials including weak subgrades.

The applicability specified in this manual is mainly between existing pavement layers and overlays and inlays to prevent reflective cracking.

They prevent or reduce the reflection in one or more of the following mechanisms:

1. Act as a barrier against crack propagation.
2. Maintain uniform load distribution over a cracked layer.
3. Provides shear resistance against rutting particularly in high-stress areas like climbing lanes.
4. Improves fatigue resistance in asphalt layers.
5. Provides additional bearing capacity.

The effect of the grid is to hold the 2 sides of a developing crack together which reduces the stress and strain at the tip of the cracks, thus reducing propagation rates.

The effectiveness of the grid depends on:

1. Stiffness of the grid
2. Geometry of the grid
3. Bond strength between the grid and asphalt

The fatigue of bound layers is determined using the stress and strain at the bottom of the layer. Asphalt reinforcement alleviates these stresses and strains and prolongs the fatigue life of the pavement.

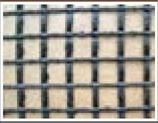
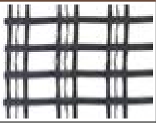
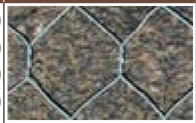
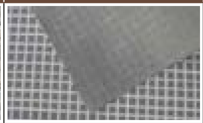
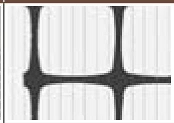
The aggregate should strike through the grid and fabric to enable adequate interlock during compaction.

Key design considerations for the choice of Asphalt Reinforcement Interlayer (ARI):

1. Minimum thickness – the minimum thickness of asphalt specified by the manufacturer of the grids or fabric must be adhered to.
2. Overall stress absorption – the ability of the ARI to absorb stress, relieve strain and provide tensile strength (carry out strength tests in the laboratory)
3. Compatibility/bond with Asphalt – the product must remain secure during operation and bond tightly with the asphalt.
4. Durability/corrosion resistance – the product must be heat resistant and stable at 205°C, resistant to biological attacks, UV radiation, weather, creep, physical abrasion and chemical degradation.
5. Milling and recycling – where milling and recycling will be required in the future, only products which can be easily milled and recycled with asphalt should be considered.
6. Boundary operating conditions/limitations/constraints – Manufacturers' recommendations should be followed.

Guidance details on the selection of reinforcement products are given in Table 8.4.

Table 8.4 Guidance on the Selection of Asphalt Reinforcement Against Reflective Cracking and Deformation in Overlay Design (SA-TG3)

	Paving Fabric	Paving Grids		Composite Paving Grids		
Issues to Consider	**Polyester or polypropylene**	**Glass fibre**	**Polyester**	**Steel mesh**	**Stitched or Warp netted**	**Bonded**
Illustrative photos						
Overlay Stress Absorption	Act as stress-absorbing interlays. Prevent ingress of water into pavement layers. Bridge shrinkage cracks. Provides increased overlay performance by 20 to 40%.	Modulus ratio of up to 20:1 over asphalt. High stiffness redirects crack energy. High stiffness resists deformation. Reduces the formation of ruts.	Increase tensile strength of asphalt layer. Reduces tensile peak stress. Assists with asphalt fatigue. Reduces the formation of ruts.	Reduces peak tensile stress. Improves asphalt fatigue. Absorbs crack discontinuities. Good rut resistance.	High stiffness redirects crack energy. Reduces peak tensile stress. Improves asphalt fatigue.	Increase fatigue life of pavement with weak foundations. Used in above application, reduces rutting and controls reflective cracking. Susceptible to creep.
Overlay Thickness	Generally, 35mm but can be as little as 25mm.	Minimum overlay thickness of 40mm 25mm overlay thickness achieved under controlled conditions.	50mm minimum. 40mm manual installation.	50mm minimum. 60mm unsupervised.	40mm minimum. 25mm used successfully in light trafficked roads with low loading.	Stiff bi-axial grids used in 70mm overlays. Thinner composite polyester grids used in 50mm overlays
Compatibility/ bond with Asphalt	Paving fabrics resistant to shrinkage. Polyester heat resistance at 210°C and performs better than polypropylenes which are sensitive at temperatures >145°C. Rough texture provides interlock adhesion. Robustness which withstands high installation damage.	Melting point 1000°C Polymer-modified bitumen coating of the grid has good compatibility with tack coat and asphalt.	Polyester heat resistance up to 210°C. Good compatibility with tack coat and asphalt.	High interlock with asphalt matrix. Tensioned and nailed at regular intervals to sub-structure.	No pre-dressing or tensioning matrix. Fabric impregnation with bitumen. The impregnated layer provides moisture-proofing. Non-woven fleece has good compatibility with tack coat and asphalt. Check the stability of reinforcement when subjected to operation heat. Glass 1000°C and polyester 260°C Polypropylenes 165°C.	No pre-dressing or tensioning required. Fabric impregnated with bitumen. The impregnated layer provides moisture-proofing. May increase pavement life by a factor of 3.
Durability and Corrosion	Polyester or polypropylenes are non-corrodible and resistant to most chemicals	Non-corrodible Resistant to oil and fuel spillage, biological attack, UV light, weather.	Non-corrodible. Resistant to oil and fuel spillage.	Steel mesh coated by bitumen when installed. Heavily zinc coated (durability).	Non-corrodible Resistance to oil and fuel spillage Thermally stable up to 165°C.	

	Paving Fabric	Paving Grids		Composite Paving Grids		
Issues to Consider	Polyester or polypropylene	Glass fibre	Polyester	Steel mesh	Stitched or Warp netted	Bonded
Milling and Recycling	Hot milling and heat scarification can cause problems. Cold milling does not usually present problems. Fabrics in excess of 150g/m³ may interfere with the milling process. Polyester fabrics are less susceptible to hot milling. Chisel teeth preferred over conical teeth. Milling speed range: 3-6m/min.	Fibre breaks down during milling and is easily recycled.	Easily milled (including hot milling) by chisel teeth and recycle.	Increase overlay thickness to allow cover during the milling operation. Asphalt milled off just above the mesh before pulling out. No recycling capabilities.	Cold milling does not present problems. Hot milling and heat scarification may cause problems where geosynthetics are present. Cognisance should be taken of the different behaviour of the paving fabric as opposed to the grid or mesh component. Chisel teeth preferred. Milling speeds of 3-6m/min. Glass fibre strands are easily mixed into new asphalt designs. Paving fabric will determine mixed design, which may contain up to 0.5% paving fabric pieces by weight.	Strong plastic grids may interfere with milling operations Aggressive milling request due to thick and hard extruded polymer strands Nonwovens milled as mentioned in Woven Paving Fabrics Recycling is unlikely as contamination of the mix is high
Boundary Operating Conditions/ Limitations and Constraints	De-lamination of the fabric could occur if: Presence of water in the base. Insufficient tack coat or saturation of the fabric. Fabric laid in rain or wet conditions. Fuel leakage or contamination between fabric and overlay. Shoving or heaving could occur due to slippage on an old, rich surface. Bleeding could occur if: Too much binder applied as a track or saturation coat. Volatile from cutback or winter grade bitumens cannot escape before applying overlay. If cut or winter grades have to be used, avoid using them in the tack coat. Mechanical failure if: Crack movement is excessive and tears fabric. Insufficient or no overlap of fabric. Laid in areas of extreme shear stress conditions. Potholes not repaired Cracks >7mm not pre-filled.	Glass grids with adhesive surfaces cannot be applied in wet conditions. Tack coat must be cured. Glass fibre is a skin irritant, so workers must wear PPE. Laid glass fibre paved same day. Sensitive to mechanical abrasion when.	Tack coat applied to dry sub-structure. Poor resistance to creep.	Inherent curvature during unrolling is removed with a rubber-tyred roller. 1st 4m securely fastened with nails or screws (1/m²). Remainder to be tensioned and fixed by nailing/ screws. Fixing in the direction of the paver. Overlap by 150mm.	De-lamination of the grid could occur due to: Presence of water in the base Insufficient tack coat or saturation of the fabric. Fabric laid in rain or wet conditions. Fuel leakage or contamination between fabric and overlay. Shoving or heaving could occur due to slippage on an old rich surface. Bleeding could occur if: Too much binder is applied as a tack or saturation coat. Volatiles from cutback winter-grade bitumen cannot escape before applying overlay. If cut or winter grades are to be used, avoid using them in the coat.	De-lamination of the grid could occur due to: Presence of water in the base Insufficient tack coat or saturation of the fabric. Fabric laid in rain or wet conditions. Fuel leakage or contamination between fabric and overlay. Shoving or heaving could occur due to slippage on an old rich surface Bleeding could occur if: Too much binder is applied as a tack or saturation coat. Volatiles from cutback winter-grade bitumen cannot escape before applying overlay. If cut or winter grades are to be used, avoid using them in the coat.

9 Structural Design of Overlays for Flexible Pavements

9.1 General

The section covers overlay design for flexible pavements, where the final pavement is considered flexible. The pavement is considered rigid if it consists of a rigid layer in the base or surfacing. This chapter covers designs, which result in a fully flexible pavement following interventions. An initially rigid pavement is considered flexible if the concrete has severely deteriorated deliberately broken in a crack and sit approach followed by a flexible overlay. For a concrete overlay on an asphalt pavement or a concrete overlay on a concrete/composite pavement, refer to Chapter 10.

In this Chapter, four overlay design methods are given in detail namely:

1. **Section 9.5**: The Catalogue Method – Involves charts where overlay thicknesses can be obtained for corresponding future traffic loading and the equivalent existing pavement surface modulus (Eq.).
2. **Section 9.6:** The Mechanistic-Empirical Design Method – Involves the determination of stresses and strains in pavement layers caused by traffic loading expressed in equivalent standard axles (ESAs) and the cumulative equivalent standard axles during the life of the pavement to the point when the failure criteria are exceeded. The design process can be carried out manually, but it is usually computer-aided using software.
3. **Section 9.7**: The AASHTO Structural Number Method – In this method, the structural number of the existing pavement and the structural number required for future traffic loading are determined. The deficit of the structural number of the existing pavement when compared with the required structural number for future traffic loading is used to determine the overlay thicknesses.
4. **Section 9.8**: Performance Method – Deflection trends are determined through routine pavement condition surveys. Generally, the deflections increase with time and so does the rut depth. A correlation is developed between the increase in rut depth and the deflections. The critical deflection corresponds to the critical rut depth (failure criteria). The overlay design aims to reset the deflection for the critical deflection to be reached after the designated design life in traffic loading or time in years. This approach is also referred to as the 'Deflection Reduction Method'.

Details are given in the relevant sections below, and an example of illustrations of the stages and expected results is given in Appendix E.

9.2 Design Principles

9.2.1 Overlay Thickness and Characteristics

No overlay structure can be designed independently of the characteristics of the overlay materials. Indeed, each overlay material has different properties and the thickness required is governed not only by the load response of the existing pavement but also by the stress/strain characteristics of the overlay itself. Consequently, this RDM 5.2 embodies one design chart/ equation for each type of overlay material, **Table 9.4**, Appendix A and Section 9.6.

9.2.2 Design Period

An overlay is designed to carry a certain number of standard axles. The estimate of the cumulative traffic loading should be calculated for the specified 'design period'.

It is known that the design period does not mean that at the end of this period, the overlaid pavement will be worn out to the point that reconstruction is required. The design period implies that towards the end of the period, the overlaid pavement will need to be strengthened again to continue to carry traffic satisfactorily for a further period.

Design periods for the overlays should normally be 20 years. However, due to budgetary limitations, a range between 7 and 15 years is considered. It is assumed that, during the design period, only routine maintenance will be carried out, i.e. shoulder and drainage system, erosion and vegetation control, localised patching and periodic resealing. This maintenance is, however, essential and if neglected will seriously affect the pavement performance and shorten its life

9.2.3 Stage Construction

Stage construction of overlays arises when it has been established that inadequate drainage is the main cause of the deterioration and that the pavement strength will be significantly increased by restoration or improvement of the drainage system and subsequent consolidation of, the saturated layers.

The first stage should then consist of:

1. Drainage restoration or improvement
2. Shoulders reinstatement, plus either resealing or flexible overlay (flexible base course covered with surface dressing)

The second stage should consist of the applying a bituminous overlay, the design of which should be based on the pavement's structural condition after drainage and consolidation.

Such stage construction will minimise the quantity of high-cost bituminous materials.

9.3 Practical and Experimental Bases

9.3.1 Use of Flexible and Bound Overlays

Flexible overlays (graded crushed stone or cement-improved materials, plus surface dressing or a thin layer of flexible premix) may be used for cumulative traffic of up to 10 million standard axles (both directions).

However, long-term considerations may, in some cases, lead to the rejection of the use of flexible overlays for medium traffic (3 MCESA -10 MCESA). Considerable thicknesses are sometimes necessary, and may cause problems with levels, junctions and shoulders.

For heavier traffic (30 MCESA – 80 MCESA), attrition and deformation would be excessive and it is therefore necessary to employ bound (semi-rigid or rigid) materials, such as asphaltic concrete, dense bituminous macadam, cement stabilised gravel or hydraulically bound stone.

9.3.2 Characteristics of Overlay Materials

9.3.2.1 Unbound Materials

The effective modulus of cohesionless materials depends to some extent, on the layer thickness and the modulus of the support (non-linear elasticity). This applies mainly to graded crushed stone. Cement or lime-improved materials generally have a higher cohesion.

It can be assumed that, within these reduced ranges of overlay thicknesses and existing pavement moduli, the overlay materials' moduli variations are limited. An average modulus can therefore be attributed to each type of material for structural calculations.

9.3.2.2 Bound Materials - Tensile Strain Criterion

When bound materials are used, the deciding criterion is generally the horizontal tensile strain at the bottom of the overlay. If this strain is excessive; the layer will crack and deteriorate rapidly.

The fatigue performance of bound materials has been estimated based on measured characteristics, field observations and theoretical considerations. See Section 9.6 for fatigue relationships.

9.3.2.3 Bituminous Mixes

Bituminous mixes are visco-plastic materials, and their dynamic moduli are therefore functions of the load application rate and the temperature.

The moduli chosen here corresponds to the following conditions:

1. Loading time: 0.02 seconds (corresponding to a vehicle speed of 60 km/h approximately)
2. Weighted mean annual air temperature: 20 °C (corresponding to the regions between 1,000 and 2,500 metres altitude) and 35 °C (for hot areas – altitude below 1000 m).

9.3.2.4 Moduli of Overlay Materials

E-Moduli for overlay materials are given in Section 6.3.3, Table 6.6, Table 6.7, and Table 6.8.

The values are for new materials.

Table 9.1 shows E-moduli ranges that have been adopted for existing pavement layers – use these values to select seeding values in the software to carry out back calculations to determine the E-moduli of existing layers.

Table 9.1 Ranges of E-Moduli of Deteriorated Existing Pavements

Material	E-Moduli Ranges (MPa)	
	Minimum	Maximum
AC in good condition	1500	3000
AC in fair to poor condition	400	1500
Granular	100	500
Subgrade	20	100

9.3.3 Construction Principles

9.3.3.1 Minimum Layer Thickness

Table 9.2 shows the minimum layer thickness below which proper laying and compaction are impractical.

Table 9.2 Minimum Allowable Thicknesses of Overlays

Materials	Minimum thickness
Graded Crushed Stone 0/40mm	125 mm
Graded Crushed Stone 0/30mm	100 mm
Treated gravel	125 mm
Treated (clayey) sand	100 mm
Dense Bituminous Macadam 0/40mm	75 mm
Dense Bituminous Macadam 0/30mm	60 mm
Dense Emulsion Macadam 0/40mm	100 mm
Dense Emulsion Macadam 0/30mm	75 mm
Asphaltic Concrete 0/20mm	50 mm
Asphaltic Concrete 0/10mm	25 mm
Sand asphalt	25 mm

9.3.3.2 Compliance with Specifications

All the materials are assumed to comply with the requirements given in Table 6.4 and all the layers are to be constructed using the current Standard Specifications for Road and Bridge Construction (SRBC).

9.4 Structural Approach

As shown in Chapters 5, 6 and 7, the empirical approach; based on the concept of allowable deflection, cannot account for the performance of rigid and semi-rigid pavements.

The structural approach is the preferred means of predicting the performance of such overlays. This approach has the further advantage of providing a good understanding of the behaviour of all types of pavements. Nevertheless, it is clear that the empirical approach remains extremely useful and that all local experience must be considered to validate the whole design method. All factual data and information obtained from overlay evaluation will have to be used, as soon as available, to check or improve the structural method proposed here.

9.4.1 Schematisation of Existing Pavement

9.4.1.1 Multi-Layer System

The existing pavement is considered an elastic multi-layer system in which the materials are characterised by Young's Modulus of Elasticity and Poisson's Ratio.

1. **Two-Layer System**

   Most of the existing pavements in Kenya have thin bituminous wearing courses (surface dressing or not more than 50 mm of asphaltic concrete wearing course) and therefore approximate reasonably to a two-layer system.

   The upper layer is characterised by its modulus $E1$, its Poisson's ratio "$\mu1$" and its thickness "h". It includes the thin wearing course, the base and, in some cases, the subbase. The subbase is to be included when its stiffness is of the same order as or greater than that of the base.

   The lower layer, taken as semi-infinite, is characterised by its modulus $E2$ and its Poisson's ratio "$\mu2$". It represents the subgrade including any improved subgrade and the subbase, when the stiffness of the latter is significantly smaller than that of the base.

2. **Three-Layer System**

   Pavements with thick bitumen surfacing (more than 50 mm) should be considered as three-layer systems.

9.4.1.2 Characterisation of the Pavement Layers

1. **Two Layer Systems**

For a standard load (geometry, dimensions and pressure) and a given value of the thickness "h" theory indicates and experience has confirmed that the product $DE2$ depends only on the ratio El/$E2$, Figure 9.1.

It has also been established that for a given value of h, the ratio R/E2 depends only on the ratio El/$E2$, Figure 9.2.

Consequently, knowledge of the deflection "D", the radius of curvature "R" and the thickness "h" enables the moduli El and $E2$ to be determined. (In addition, the subgrade moduli can be measured directly by plate bearing test), Figure 9.3.

The relationship between $DE2$, $R/E2$ and El/$E2$ are plotted in Figure 9.3 and Figure 9.4. These relationships are based on Burmeister's theory. In this method, the following assumptions are made:

a. The standard load represents the dual wheel assembly of a 13-tonne axle. It is assumed to be equivalent to a vertical pressure "q" uniformly distributed over two equal circular areas of radius "a", the centres of which are at a distance "I" apart where: -

q = 6.62 kg/ cm^2

a = 125 mm

I = 3a = 375 mm

13 tonne should be used as reference load and if any other loading is used in the test the values must be converted to the reference load using Equation 9.7.

b. The Poisson's ratio of all materials is equal to 0.25.

It appears that, for thicknesses "h" greater than 100 mm and ratios $E1/E2$ smaller than 50 (virtually all Kenyan roads due for strengthening satisfy these conditions), the curves of charts given in Figure 9.1 to Figure 9.4 can be represented, with sufficient accuracy, using Equations 9.1 to Equation 9.7:

$$D.E2 = \frac{11600}{\left[\frac{E1}{E2}\right]^Y}$$

Equation 9.1

$$\frac{R}{E2} = \frac{0.028}{\left[\frac{E1}{E2}\right]^X}$$

Equation 9.2

In which the exponents X and Y are expressed as follows:

$$x = 0.860\ log\ h - 0.474$$

Equation 9.3

$$y = 0.493\ log\ h - 0.410$$

Equation 9.4

By combining equations Equation 9.1 and Equation 9.2 the following relationships, giving El and ($E1/E2$) as functions of "D", "R" and "h" are obtained, Equation 9.5:

$$[E1]^R = \frac{R}{0.028} \text{ x } \left[\frac{D}{11600}\right]^A$$

Equation 9.5

in which:

$$A = \frac{1 - X}{1 - Y} \quad \text{and } B = 1 - A$$

$$\left[\frac{E1}{E2}\right]^{X\text{-}Y} = \frac{R \text{ x } D}{3248}$$

Equation 9.6

The charts correspond to a 13-tonne axle, whereas deflection tests in Kenya were carried out using a 6.3-tonne axle. Equation 9.6 is therefore modified accordingly and becomes, Equation 9.7:

$$[E1]B = \frac{R}{0.056} \text{ x } \left[\frac{D}{58000}\right]^A$$

Equation 9.7

Equation 9.6 remains unchanged.

The values of $E1$ and $E1/E2$ can be obtained from the two equations.

2. Three-Layer System

Determination of the 3 moduli from deflection and radius of curvature is not possible. Moduli must therefore be measured individually either in-situ by plate-bearing, tests or dynamic methods or, in laboratory tests on Core samples.

1
2
3
4
5
6
7
8
9
Structural Design of Overlays for Flexible Pavements

3. Four-Layer System

For both 3-layer and 4-layer systems or more it is recommended to use modern software for back-calculating the E-moduli of each layer as is described in Section 9.6.

Note:
It is necessary to verify that the theoretical model of the existing pavement derived from the elastic method gives a valid representation of the actual structure. This is why several different tests are desirable on each layer, e.g. field density and moisture content, grading and plasticity, UCS, etc. as indicated in Chapter 6.

Figure 9.1 Product of DxE2 vs E1/E2

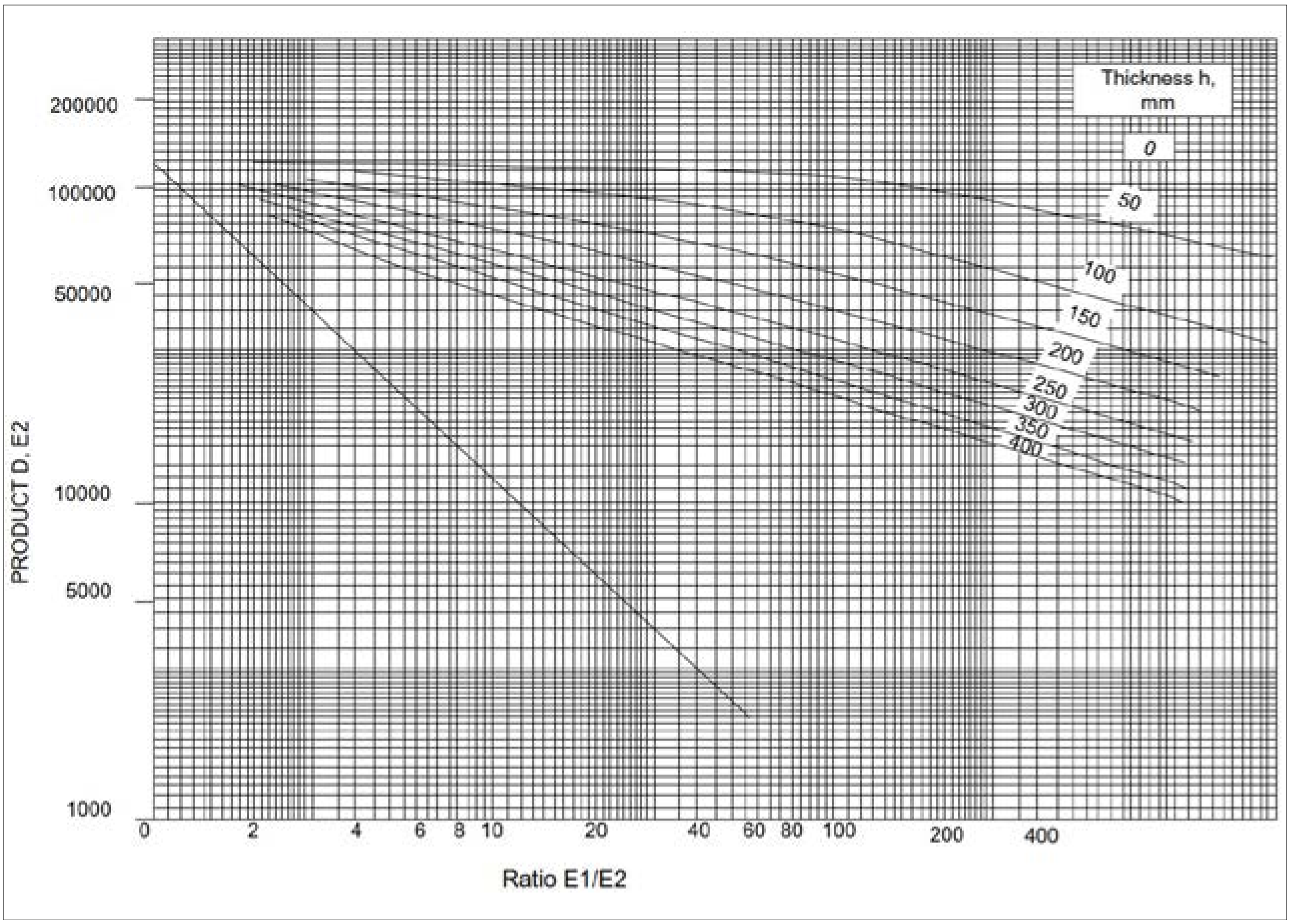


Figure 9.2 Product E1xE2 vs E1/E2

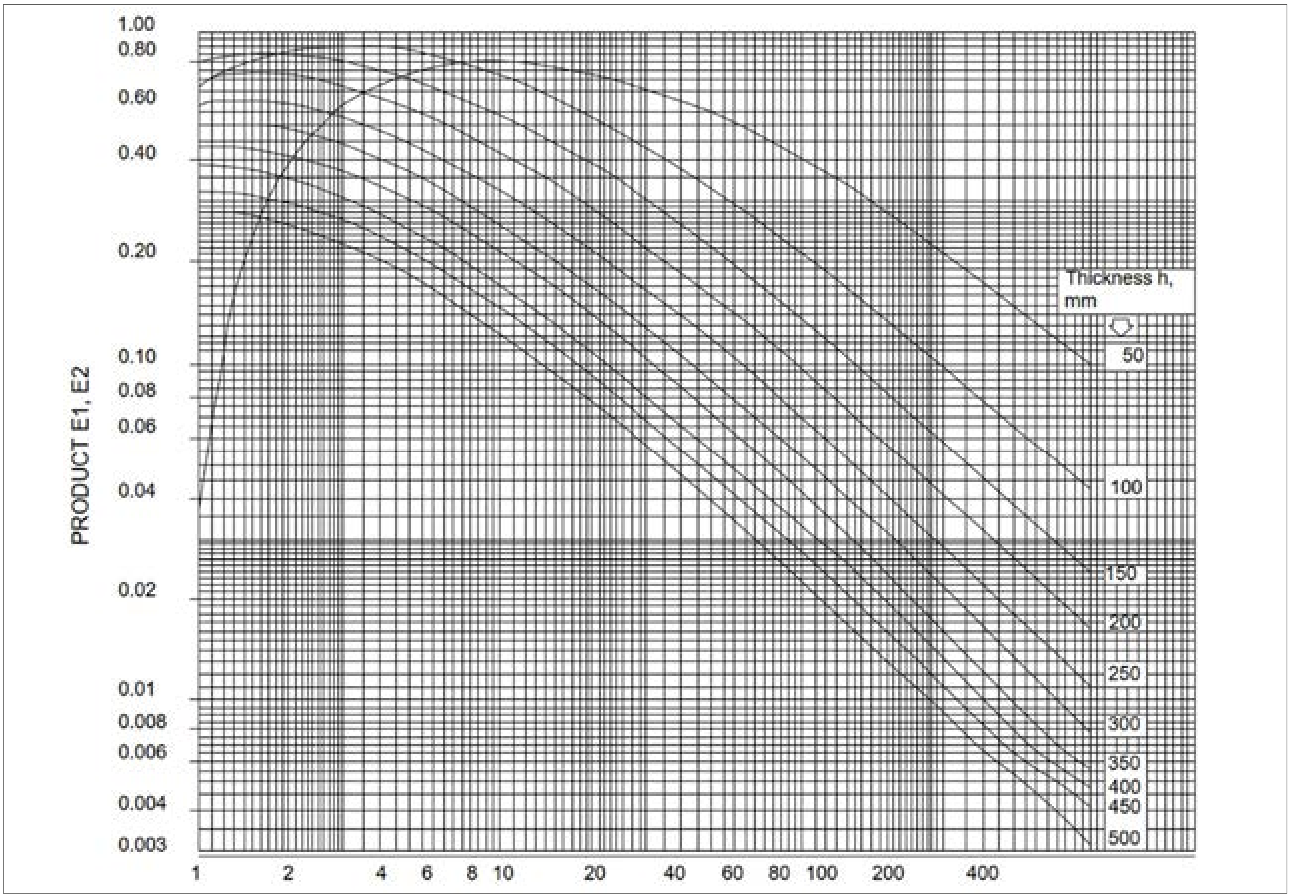


Figure 9.3 Ratio R/E1 vs E1/E2

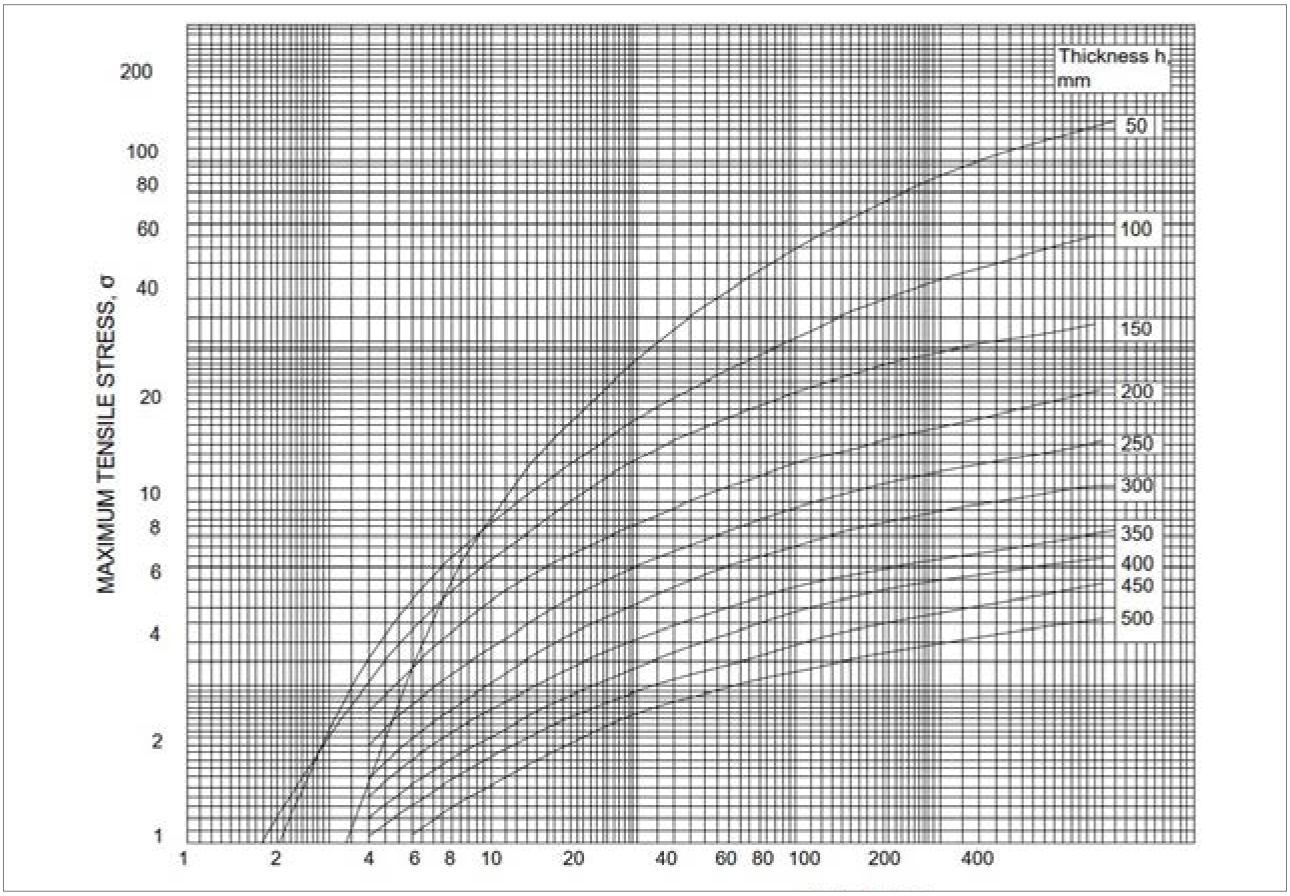
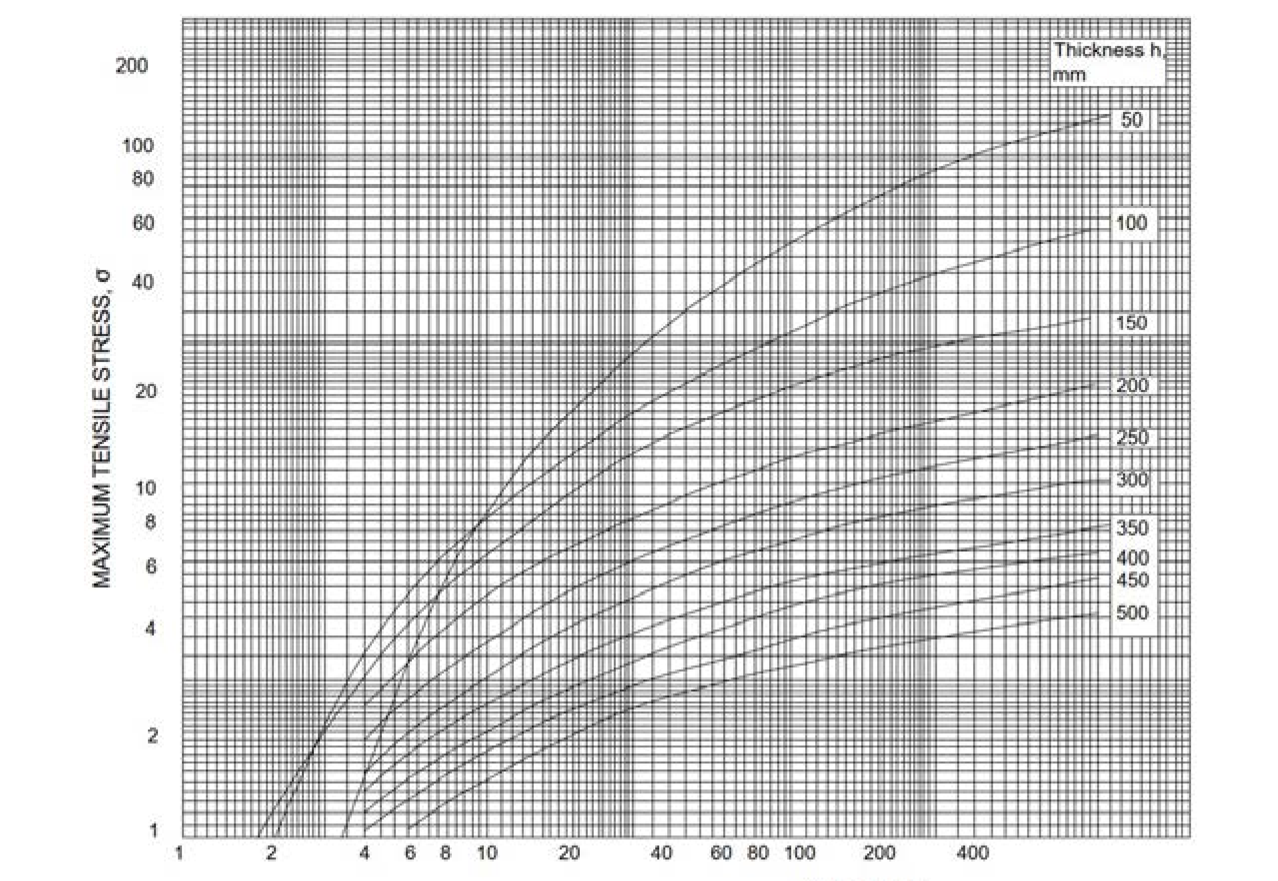


Figure 9.4 Maximum Tensile Stress vs E1/E2



9.4.2 Pavement's Equivalent Modulus

In practice and for overlay design, the existing pavement is characterised by an 'Equivalent Modulus'. This equivalent modulus is defined as the modulus of a hypothetical one-layer system such that an overlay placed on it would be subjected to the same stress and strain when placed on the existing pavement and subjected to the same load. Equivalent modulus is an abstract notion and should not be confused with any of the physical moduli, which can be attributed to a material.

Using the elastic method described above it has been established the equivalent modulus Eq. of a homogeneous section of a pavement can be expressed by Equation 9.8 and Equation 9.9:

$$Eq = 10^{a-1} \text{ x } E1 \text{ x } \left[\frac{R}{D}\right]^{a-2}$$

Equation 9.8

Where 'E1' is the modulus of the upper layer of the existing pavement.

'D' and 'R' are the characteristics values of the homogeneous section considered (i.e. D90 and R10) and

'a' is a coefficient calculated from the formula

$$'a' = 1/[1 + log_{10}\left[\frac{E1}{1018}\right]]$$

Equation 9.9

By combing Equations 9.8 and Equation 9.9, the equivalent modulus Eq. can be expressed as a function of the parameters 'h', 'D' and 'R'.

9.4.3 Calculation of Stress and Strain

The overlaid pavement is considered an elastic two-layer system, in which the materials are characterised by their moduli and Poisson's ratio, the exiting pavement being typified by its "equivalent modulus".

The calculation of stress and strain is carried out based on the following mechanistic assumptions:

1. The design load is assumed to be uniformly distributed over two equal circular areas.
2. The pavement and overlay materials' volumetric changes to the load are based on Poisson's ratios given in Section 6.3.3.
3. All layers are considered to have complete friction between them.

The method allows the following calculations to be made:

1. The horizontal tensile stress and strain at the bottom of overlays from bound material.
2. The vertical compressive stress and strain on the surface of the existing pavement.

9.4.4 Determination of Required Overlay Thickness

9.4.4.1 Flexible Overlays

In the case of flexible overlay materials, the deciding criterion is the compressive vertical strain on the surface of the existing pavement or the surface of the subgrade. The thickness of the flexible overlay required is determined by comparing the calculated compressive strain with the maximum permissible strain, as deduced from experience and comparison with the standard structures of the RDM 3.

9.4.4.2 Bound Overlays

In the case of bound overlay materials, the deciding criterion is the tensile strain at the bottom of the overlay. The thickness of the overlay required is therefore determined by comparison of the calculated tensile strain with the maximum permissible strain as deduced from the strain-life relationship of the material considered.

The strain-life relationship of the main bound overlay materials considered (asphaltic concrete type I, dense bituminous macadam, dense emulsion macadam, hydraulically bound stone and hydraulically bound gravel) have been derived from measured characteristics, monitoring of the performance of overlay trial sections and theoretical considerations. The corresponding fatigue equations are given in Table 9.4. This should be used in conjunction with the fatigue equations in Section 9.6.

In this respect, the maximum tolerable strains chosen do not correspond to the initiation of fatigue cracking at the bottom of the layer, but to the appearance of the first cracks at the surface. In other words, the time of propagation of fatigue cracking through the layer has been included in the design life in accordance with mechanistic theory. However, generally, cracking starts from the top due to environmentally induced deterioration of the bituminous surfacing/wearing course through oxidation and embrittlement..

9.5 Catalogue Method

For each main type of the overlay material envisaged, the thickness required has been calculated by the structural method presented in Section 9.4, as a function of the cumulative traffic and the equivalent modulus of the existing pavement and effective structural number (SNP/SN_{eff}) respectively. The results are plotted in seven overlay design charts listed in Table 9.3 given in Appendix A.

1
2
3
4
5
6
7
8
9
Structural Design of Overlays for Flexible Pavements

Table 9.3 List of Charts/Catalogues of Overlay Design Using Equivalent Modulus

Catalogue No.	Overlay
1	Asphaltic Concrete Type II, and Type I + Surface dressing
2	Dense Bituminous Macadam + ACI +Surface dressing
3	Bitumen Stabilised Material + ACI +Surface dressing
4	Hydraulically bound stone (HBS9) + ACI +Surface dressing
5	Hydraulically bound stone (HBS6) + ACI +Surface dressing
6	Hydraulically bound stone (HBS3) + ACI +Surface dressing
7	Graded Crushed Stone + Surface dressing

9.5.1 The Choice of the Cheapest Solution

The selection of one type of overlay is mainly a matter of cost and availability of materials. In this respect, the following points are emphasised.

Asphaltic concrete is suitable for all traffic, but its exclusive use is generally uneconomical when the thickness exceeds 120 mm:

1. Dense bituminous macadam and hydraulically bound stone are unlikely to prove economical for traffic lighter than 10 MCESA.
2. Dense emulsion macadam as an overlay material has several advantages, and should often prove a viable and economical alternative to dense bituminous macadam and hydraulically bound stone.
3. Hydraulically improved gravel compares favourably with other materials in strengthening very deformable pavements.
4. Flexible overlays (GCS or cement-improved materials) are limited to traffic loading less than 10 MCESA. Moreover, long-term considerations may sometimes eliminate flexible overlays for medium traffic (TC3) as explained in Section 8.4.
5. When the combinations of materials used for surfacing and base are fully flexible the materials above including GCS can be used for higher traffic loading > TC10 ideally up to TC80.

9.5.2 Other Types of Overlays

The choice of an overlay is not restricted to one of the 7 main types considered above.

Consideration may well be given to other overlay materials, as suggested in Section 8.4.

On the other hand, the design of an overlay may be influenced by factors other than permissible strains, e.g. prevention of reflection cracking.

9.5.3 Method of Use

1. **First Step:** Determination of the Design Parameters.

   Evaluation of the pavement will enable homogeneous sections to be defined, as explained in Chapter 6.

   For each homogeneous section, the design parameters, i.e. the characteristic deflection $D90$ and the characteristic radius curvature $R10$ should be determined. The thickness of the upper layer of existing pavement should be measured and, if necessary, averaged.

2. **Second Step:** Schematisation of Existing Pavement.

   For each homogeneous section, the moduli $E1/E2$ and Eq. of the existing pavement are determined by calculations or graphical means.

   For the former, moduli and overlay thicknesses are derived by the following equations and relationships: (Figure 9.1, Figure 9.2, Figure 9.3 and Figure 9.4). Equation 9.10 and Equation 9.11 are used to calculate stress and strain.

$$et \times E2 = \frac{2.39 - 1.35 \log h}{1.58 \left[\frac{E1}{E2} \right]^{h}}$$

$$\text{with } b = 0.27[1 - e^{-\log h}] \log \left[\frac{E1}{E2} \right]$$

Equation 9.10

$$\log dt = \frac{[1.66 - 10f] \times [1.84 \left[\frac{E1}{E2} \right]^{0.0194} - \log h]}{1.58}$$

Equation 9.11

$$f = -0.856 \log [\left[\frac{E1}{E2} \right] + 15] + 0.803$$

Where "*et*" and "*dt*" are permissible stress and strain for the particular design traffic.

The moduli of various materials are presented in the section 6.3.4.2.

Fatigue laws for the overlay material considered are given in Table 9.4:

Table 9.4 Fatigue Laws for Different Types of Overlay Materials

Material	Moduli (MPa)	Fatigue Laws
Asphaltic Concrete (Type II)	2500	$et = 10^{-0.21 logT - 2.2325}$
Asphaltic Concrete (Type I)	4000	$et = 10^{-0.20 logT - 2.337}$
Dense Bitumen Macadam	5000	$et = 10^{-0.25 logT - 2.158}$
Dense Emulsion Macadam	2000	$et = 10^{-0.17 log\ T - 2.430}$
Hydraulically bound stone (HBS9)	10000	$dt = -2.0\ logT + 20.6$
Hydraulically bound stone (HBS6)	5000	$dt = -0.75\ logT + 8.14$

3. **Third Step:** Estimation of Cumulative Traffic.

As detailed in the Pavement Design Manual for New Roads, this Step involves the following operations:

   a. Studying the initial traffic flows.

   b. Studying axle-load distribution.

   c. Choosing the design period.

   d. Estimating the traffic growth rate.

   e. Calculating the cumulative number of standard axles.

4. **Fourth Step:** Inventory and Study of the Available Overlay Materials and Selection of the Possible Types of Overlays.

   Knowledge of the types and characteristics of the possible overlay materials, traffic and the peculiarities of the existing pavement will permit the selection of one or more types of overlay.

5. **Fifth Step:** Determination of the Overlay Thickness Required.

   The thickness required for each type of overlay should be obtained from cumulative traffic and equivalent modulus, using the appropriate overlay design chart.

6. **Sixth Step:** Economic Comparison of the Possible Overlays and Final Choice of One Overlay Structure.

   The total cost of each possible overlay should be estimated considering all ancillary works, such as levelling course, shoulders upgrading, deviations etc.

   In this respect, the importance of the problem of handling traffic during construction is stressed, particularly as the construction and maintenance of deviations may be very costly.

7. **Finalising the Overlay Design and Preparation of Special Specifications.**

   Finally, this economic comparison should enable the design engineer to select the optimum solution.

1 2 3 4 5 6 7 8 9 Structural Design of Overlays for Flexible Pavements

8. **Seventh Step:** Finalising the overlay Design and Preparation of Special Specifications.
   For the final refinements to the overlay design, consideration should be given to every peculiarity of the project (materials, climate, drainage, road safety, etc.) and special specifications shall be prepared.

9.5.4 Catalogues

Following the calculation of the equivalent modulus and traffic loading, the overlay thickness is then determined using the Catalogues given in Appendix A. The design engineer should consider the following while using the catalogues:

1. Thick overlays are not recommended for traffic loading ≤ 1 MCESA.
2. Where thick AC is required, it is recommended to use DBM/DEM for the binder course and with surface dressing or thin AC surfacing.

9.6 Mechanistic-Empirical Design Method

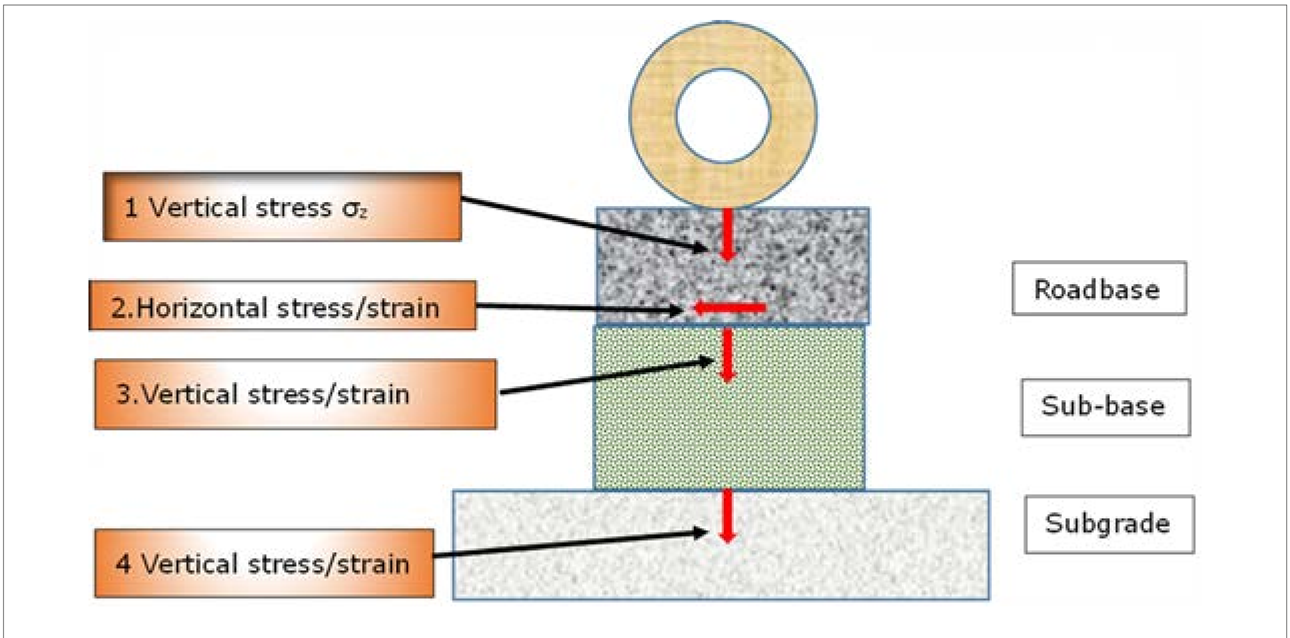
This section covers the mechanistic-empirical design method in determining overlay thicknesses for a multi-layer pavement system. The road pavement is a system of linear elastic layers. In mechanistic design, the following assumptions are made:

1. Layers are linearly elastic.
2. The material making up the layers is homogeneous and isotropic.
3. Subgrade has infinite depth.
4. Pavement layers are infinite longitudinally and semi-finite in the transverse direction.
5. No slippage occurs during deflection.

These assumptions are not necessarily true because the layers are generally not linearly elastic, and materials are by their nature, highly variable to be homogenous or isotropic. Variability is therefore expected in the results developed based on these assumptions.

The design principles are based on the determination of critical stresses and strains of the existing road under load as illustrated in Figure 9.5 and that every pass of an equivalent standard axle at a particular point consumes the life of the pavement. The life of the pavement is therefore the number of repetitions of the standard axle that would cause the failure of the pavement.

Figure 9.5 Illustration of the Stresses and Strains Considered in Mechanistic Design



The mechanistic method involves:

1. Surface condition evaluation covered in Section 6.2.
2. Characterisation of the existing pavement and materials covered in Section 6.3.3. and Section 6.3.4.
3. Determination of the pavement design life to carry future traffic.
4. Determination of the appropriate scheme of overlay.
5. Determination overlay thickness(es) of the layer(s) for the overlay.
6. Economic evaluation of the solution options .
7. Finalisation of the overlay design.
8. Preparation of the design report.

9.6.1 Surface Condition Evaluation

Surface condition is a determinant factor in deciding on which intervention to adopt. Key considerations include:

1. The present serviceability index (PSI).
2. Present serviceability Rating (PSR).
3. International roughness index (IRI).
4. Rut depth (mm).

The thresholds for determination of interventions required, whether it should be maintenance or rehabilitation design (overlay design) are covered in Chapter 7 – Information shall be provided as illustrated in Table 9.5. This information covers the functional aspects of the evaluation of the pavement.

Table 9.5 Format for Decisions on Rehabilitation/Maintenance Based on Surface Condition

Item No.	Surface Condition Parameter	Threshold	Value	Remarks: Rehabilitation/ Maintenance Intervention
1.	Present serviceability index (PSI)			
2.	Present serviceability rating (PSR)			
3.	International roughness index (IRI)			
4.	Pavement Condition Index (PCI)			
5.	Rut depth (mm)			

1
2
3
4
5
6
7
8
9
Structural Design of Overlays for Flexible Pavements

9.6.2 Characterisation of Pavement Layers and Subgrade

Record the E-moduli, thicknesses of the pavement layers and their depth for each uniform section. Further information regarding the characterisation of pavement layers is given in Table 6.4 for materials properties, Table 9.1 for E-moduli for deteriorated pavements, and Table 6.6, Table 6.7 and Table 6.8 for Poisson's ratios and E-moduli for new materials.

It is important to note that engineering knowledge and experience are required to assign E-moduli for deteriorated layers. If the analysis is not carried out entirely using software, the design calculations can be carried out following the format illustrated in Table 9.6.

Table 9.6 Illustration of Calculation of Corrected E-modulus for Design

Item No.	Chainage	Layer	Thickness	E-modulus	Temp. correction	Seasonal correction	Corrected E-modulus	Poisson's ratio
1.	e.g. 1+000	WC						
		DBM						
		GCS						
		SB						
		SG		Modulus of Resilience (M_R)				

The moduli calculated from the back-calculation at different temperatures should be corrected to a reference temperature. For tropical regions like the Coast, North and North Eastern Kenya, use 35°C as reference temperature (Equation 9.12). For subtropical regions like Nyanza, the Highlands, and Western Kenya, use 20°C as reference temperature (Equation 9.13). Use these relationships (the Reddy equation, 2003) for temperature correction of the E-modulus.

$$E_{T1} = fE_{T2}$$

Equation 9.12

$$f = \frac{1 - 0.238LnT_1}{1 - 0.238LnT_2}$$

Where,

E_{T1} = Modulus at the reference temperature, T_1.

E_{T2} = Modulus the temperature measured during the test, T_2.

f = Temperature correction factor for layer moduli.

The equation is valid for a temperature range of 25 °C to 40 °C.

For temperature correction using a 20 °C to 25 °C reference use the equation below.

$$E_{20} = E_T \text{ x } 10^{(0.0003 \text{ x } (20 - T)^2 - 0.002 \text{ x } (20 - T))}$$

Equation 9.13

Where,

E_{20} = Stiffness at 20 °C.

E_T = Stiffness at temperature T.

T = Temperature of the asphalt (at a depth of 100 mm) at the time of testing.

9.6.3 Determination of Residual Life of the Pavement and Subgrade

Determination of residual life of the pavement i.e., the number of remaining cumulative passes or repetitions of the equivalent standard axle loads (CESAs).

The analysis requires the determination of critical stress and strain.

1. For bound layers, calculate the tensile stress (σ_t) and corresponding strain ($\mathcal{E}_t$) at the bottom of the layer resulting from the loading. Cracks are bound to develop at the position of the highest tensile stresses and strain and could be at the bottom of the layer causing bottom-up cracking. The highest tensile stresses and strains could also emanate from the top causing top-down cracking which is more common. However, cracking shall be considered bottom-up and referred to as fatigue cracking.
2. For unbound and slightly bound or modified materials calculate the stress (σ_v) and strain ($\mathcal{E}_v$) at the surface of the layer. It is assumed that the strain that occurs is not fully elastic all the time, and that minute creep or deformation, always occurs causing cumulative deformation. This will show in the form of rutting in the wheel path. The deformation may occur in any of the pavement layers or the subgrade. Deformation is considered to be predominant for unbound and slightly bound layers.
3. Determine the values of critical stress and critical strain for the different layers. This can be carried out manually using the multi-layer elastic theory.
4. Determine the surface modulus at the top of the pavement is calculated using Equation 9.14 and Equation 9.15.

$$E_0 = 2(1 - \mu^2)\,\sigma_0\,\frac{a}{\delta_0}$$

Equation 9.14

5. Determine the surface modulus at equivalent depth (r):

$$E_0(r) = (1 - \mu^2)\,\sigma_0\,\frac{a^2}{r.\delta_r}$$

Equation 9.15

Where,

E_0 = Surface modulus at the centre of the loading plate (MPa).

$E_0(r)$ = Surface modulus at distance r (MPa).

μ = Poisson's ratio.

σ_0 = The contact pressure under the plate.

a = Radius of the loading plate.

δ_r = The deflection at a distance r (microns).

6. Use software to determine the vertical stresses and vertical strains then the tensile stresses and strains. Enter the required parameters into the software such as:
   a. E-moduli, thicknesses, Poisson's ratios, load parameters, etc.
   b. Apply the load parameters i.e., the wheel load of consideration. This can be the individual ESAs or the representative load for a traffic category.
   c. Analyse to determine the stresses and strains depicted in Figure 9.5 for the different bound and unbound layers.
   d. Apply the fatigue laws, which are the relationships between the load repetitions and critical strain to determine the remaining or residual fatigue life of the existing pavement. This could be in the form of a fatigue line or fatigue equation which are the failure criteria, see Equation 9.16 to Equation 9.26.

1
2
3
4
5
6
7
8
9
Structural Design of Overlays for Flexible Pavements

9.6.3.1 Fatigue Failure Criteria

1. Asphalt failure criteria. The applicable ones for Kenya are as follows:
   a. For ACI (High stability asphalt/Superpave 4000 MPa),

$$\varepsilon_t = \frac{4710}{N^{0.202}}$$

Equation 9.16

   b. For ACII (Flexible asphalt 2500 MPa),

$$\varepsilon_t = \frac{5855}{N^{0.207}}$$

Equation 9.17

   c. For DBM (DBM/Superpave DBM 5000 MPa),

$$\varepsilon_t = \frac{6958}{N^{0.250}}$$

Equation 9.18

   d. For Bitumen Sand Asphalt (1000 MPa),

$$\varepsilon_t = \frac{5853}{N^{00.193}}$$

Equation 9.19

Where,
N = The design traffic in equivalent standard axles.
ε_t = The horizontal microstrain at the bottom of the bituminous layer.

   e. EME asphalt mixtures (8000 MPa)

$$N = F^* \left[\frac{57500}{\mu\varepsilon \times S_{mix}^{0.36}}\right]^{5.5}$$

Equation 9.20

Where,
N = The design traffic in equivalent standard axles.
S_{mix} = Elastic modulus of the mixture in MPa.
$\mu\varepsilon$ = Horizontal microstrain in the asphalt.
N = Number of strain repetitions to failure.
F = Calibration factor (1-4).

   f. Other bituminous mixtures,

$$N = F^* \left[\frac{6918 \times (0.856 \times V_b + 1.08)}{\mu\varepsilon \times S_{mix}^{0.36}}\right]^5$$

Equation 9.21

Where,
N = The design traffic in equivalent standard axles.
S_{mix} = Elastic modulus of the mixture in MPa.
$\mu\varepsilon$ = Horizontal microstrain in the asphalt.
N = Number of strain repetitions to failure.
V_b = Proportion of bitumen by volume in the mixture, as a %.
F = Calibration factor (1-4).

5. Failure criteria for hydraulically bound materials. The applicable ones for Kenya are as follows:
   a. For Hydraulically Bound Stone (HBS9) (10000 MPa),

$$\sigma_t = \frac{4350}{N^{0.117}}$$

Equation 9.22

   b. For Hydraulically Bound Stone (HBS3) (4000 MPa),

$$\sigma_t = \frac{1700}{N^{0.111}}$$

Equation 9.23

Where,

N = The design traffic in equivalent standard axles.

σ_t = The horizontal stress at the bottom of the bound layer (measured in kPa).

   c. For other hydraulically modified or bound stone (e.g., HMS1 and HBS6)

$$N = \frac{2^{10}}{e^{0.011\sigma_t}}$$

Equation 9.24

Where,

N = The design traffic in equivalent standard axles.

σ_t = The horizontal stress at the bottom of the bound layer (measured in MPa).

e = The natural constant whose value is 2.71828.

6. Subgrade failure criteria. The applicable ones for Kenya are as follows:
   a. For Low-Volume Sealed Roads

$$\varepsilon_c = \left[\frac{41445}{N^{1/4}}\right]^4$$

Equation 9.25

   b. For all pavements designed to carry more than 1 MESA,

$$\varepsilon_c = \frac{28000}{N^{1/4}}$$

Equation 9.26

Where,

N = The design traffic in equivalent standard axles.

ε_c = The vertical microstrain at the top of the subgrade.

The **residual life** of the existing pavement is the lowest of the residual lives calculated of all the pavement layers and the subgrade.

1
2
3
4
5
6
7
8
9
Structural Design of Overlays for Flexible Pavements

9.6.4 Determination of Design Life

Design life for rehabilitation and overly design is a decision a design engineer should make considering the following factors:

1. **Road standard** (based on road classes) – consideration should be made based on whether the road is a low-volume or medium-trafficked or high-volume road (highway or expressway). Guidance is given in Table 9.9.
2. **Available Resources** – it is important to assess the resources available to determine economically feasible intervention solutions. The engineer should consider a staged construction approach to suit the available budget. This can be achieved by reducing the design life to reduce the costs of rehabilitation and overlay.
3. **Purpose of rehabilitation** – the condition of the road is an important determinant factor. If the road is in bad condition an interim solution may be required. The interim solution may involve maintenance activities to repair surface defects and apply a thin overlay or reseal design for a much shorter design life, which may be inadequate for future traffic but appropriate for the available budget. Conversely, for a road in good condition, rehabilitation may be necessitated by a change of functionality or an unplanned increase in traffic, e.g., diverted traffic. Interim strengthening may be required which may involve a thin overlay.
4. **Long-life pavements** – a decision may be made to turn the existing pavement into a long-life pavement. The design life may be decided based on CESAs rather than years. Generally, a design for 80 MCESA yields long-life pavements. No further increase in the thickness from the 80 MCESA design will aid pavement longevity and will therefore be redundant extra strength and thus unnecessary additional cost. If the traffic is expected to remain light to very light, a lower standard than 80 MCESA will result in a long-life pavement. Guidance is given in Table 9.7.

Table 9.7 Guidance on Long-life Pavements

Item No.	Standard of Road	Maximum Microstrains Limit for Long Life Pavements	Standard of Design	Engineering Considerations
1	High Volume Roads (HVRs)	Subgrade/ foundation < 200 Base and AC < 70	80 MCESA	Ensure the possibility for in-lays to replace deterioration of the surfacing. Ensure that the base is strong enough and has the required longevity.
2	Low Volume Roads (LVRs) 1 MESA max.	Subgrade/ foundation < 200 Base and AC < 70	3 MCESA	Schedule resealed to prevent deterioration of the surface of the roads especially thin bituminous surfacings.

5. The designer should also provide the associated future maintenance interventions that are appropriate for the stipulated design life. Table 9.8 provides guidance on such interventions. These may include a schedule for crack sealing before the beginning of the rainy season, fog sprays every 4-5 years, reseals or micro surfacing every 7-10 years, or application of seals on AC wearing course accordingly.

Table 9.8 Schedule for Future Maintenance Interventions

Item No.	Intervention	Recommended Schedule	Key engineering considerations
1	Crack sealing	Before every rainy season	This shall be standard practice in pavement maintenance
2	Fog spray	After every 4-6 years	This shall be scheduled as recommended or responsive to fine cracks development, spalling and embrittlement of the bituminous surfacing.
3	Micro-surfacing	After every 6-8 years	Schedule as recommended or at crack initiation. Apply surface treatment
4	Reseals	After 7-10 years	Schedule as recommended or at crack initiation. Intervention is appropriate for all classes of road.
5	Thin comfort overlays	After 10-15 years	Schedule as recommended when roughness has increased due to minor deformations, corrugations, and widespread patching. Appropriate for medium and high-volume roads.

9.6.5 Calculation of the Design Traffic Loading

Design traffic loading should be determined reasonably accurately. Underestimating design traffic leads to accelerated deterioration or premature failures. Overestimating traffic loading leads to overdesign and unnecessarily higher costs, which may impact the project negatively. More details are given in Chapter 5, and RDM 5.1 and RDM 3.

9.6.6 Design of Overlay Thickness

The overlay is a means of strengthening the pavement to increase its capacity to accommodate additional repetitions of the ESA over and above the allowable number i.e., the remaining/residual life. This is the difference or deficit between the design life and residual life.

The procedure involves:

1. Assessment of the difference between the design life and residual life and intervention decisions are covered in Table 9.9.

Table 9.9 Guidance on Rehabilitation Decision Option

Item No.	Comparison Between Design Life and Residual Life	Rehabilitation Decision
1	Design life ≤ residual life	No overlay is required design for maintenance to improve surface condition. Apply a reseal or a thin comfort AC overlay. If surfacing is in good condition with few single fine cracks, apply fog sprays or micro-surfacing or reseal.
2	Design life > residual life	Select the overlay option and design the thickness of the overlay. Consider preparation as well as repairs on the existing pavement and surface. Design the overlay thickness. If the surface is badly cracked consider designing a crack control mechanism, which may include applying a crack control layer (granular material) or geogrid or other reinforcement. Select the overlay option and design the thickness of the overlay. Consider preparation as well as repairs on the existing pavement and surface.

2. Consider options for the types of overlays to use. Guidance on this is given in Table 9.10.

Table 9.10 Guidance on Types and Options for the Overlays

Overlay option	Overlay type
1	AC type 1
2	AC type 1 on DBM
3	AC type 1 on DBM and crack control layer (GCS)
4	AC on GCS
5	DSD/TSD/Cape seal on GCS
6	AC on Macadam base
7	AC on Telford base (Hand Packed Stone)
8	Hydraulically bound stone
9	Reinforced concrete
10	DSD/SD on OGPA
11	DSD on hydraulically bound base
12	AC on hydraulically bound base

3. Calculation of the thickness of the overlay is an iterative process. At this stage, the pavement system is the layers of the existing pavement plus the overlay which may consist of one or more layers. The strength characteristics (E-moduli) of the existing pavement layers, the subgrade, and the overlay layers are known. The thicknesses of the existing pavement layers are known, and the thicknesses of the overlay layers are then assumed. The iteration can be done manually but it is cumbersome. Several software packages are available to carry out the iteration.

   a. Using the new layer system, follow the processes for determining the micro stresses and micro-strains of the new layer system ($\mathcal{E}_v$ and $\mathcal{E}_t$) for all the layers i.e. $\mathcal{E}_t$ for bound layers and $\mathcal{E}_v$ for unbound layers.

   d. Using the micro strains determine the new life of the pavement i.e., existing + the overlay.

   e. Compare the residual life of the pavement with the design life, see guidance in Table 9.9. If the design life (CESA) is greater than the life of the new pavement, increase the thickness of the overlay and repeat the analysis. Continue the iteration until the life of the pavement is greater than the design life in CESA.

   f. Try two more options of overlays and carry out the iteration until the design life is less than the life of the pavement.

   g. Estimate the life cycle costs for each option. Standard unit costs can be used for this purpose.

   h. Prepare a plan for future interventions including routine and perioding maintenance.

   i. Sum up the capital and recurrent costs to determine life cycle costs.

   j. Carry out an economic appraisal of the 3 options and select the most economical one.

9.7 The AASHTO Structural Number Method

The structural number SN is a value that represents the strength of pavement layers in their compacted state. The value depends on the strength of the material and the thickness of the layer. The strength of the pavement is the summation of the strength contributions of the different layers. The AASHTO Structural Number Method for pavement rehabilitation design is based on this engineering principle.

9.7.1 Determination of Pavement Structural Number

The structural number (SN) of a pavement layer is calculated using the following AASHTO Equation 9.27:

$$SN = 0.0394mah$$

Equation 9.27

The structural number of the pavement is calculated using the Equation 9.28 below:

$$SN = 0.0394 \sum_{i}^{k} m_i \, a_i \, h_i$$

Equation 9.28

Where,

a = Layer strength coefficient.

h = Thickness of the layer, mm.

m = A drainage factor, generally considered as 1.

The structural design involves determining of an appropriate structure and hence the strength of pavement that is adequate to carry the traffic loading i.e., the repetitions of the equivalent standard axle (ESA), see Chapter 5 for further details on the calculation of ESAs. The number of repetitions that can be carried to the point of structural failure is the life of the pavement. The failure is defined by confidence/reliability i.e., 90 % reliability for HVRs and 80 % reliability for LVRs. This means that at the end of the life of the pavement, only 10 % and 20 % of the pavement respectively would have failed.

9.7.1.1. Modified Structural Number

SN is generally a summation of SNs of the pavement layers only. The structural number should be adjusted to include the contribution of the subgrade, and this is the modified structural number SNC. SNC is calculated using the following Equation 9.29.

$$SNC = SN + 3.51\,(Log_{10}CBR_{sG}) - (Log_{10}\,CBR_{sG})^2 - 1.43$$

Equation 9.29

Where,

$CBRSG$ = The CBR of subgrade.

SN = Structural number of the pavement (summation of all pavement layers).

9.7.1.2 Adjusted Structural Number

The structural number of each layer depends on its contribution to the overall strength of the pavement which in turn is dependent on its depth from the surface. This would be the adjusted structural number (SNP). SNP is calculated using Equation 9.30:

$$SNP = SNA + SNS + SNG$$

Equation 9.30

Component terms are calculated using Equation 9.31, Equation 9.32 and Equation 9.33;

$$SNA = 0.0394\sum_{i=1}^{n} a_i \, h_i$$

Equation 9.31

$$SNS = 0.0394\sum_{j=1}^{m} a_j\left\{\left[\frac{b_0\, exp(-\,b_3\, z_j)}{-\,b_3} + \frac{b_1\, exp(-(b_2 + b_3)z_j)}{b_2 + b_3}\right] - \left[\frac{(b_0\, exp(-b_3\, z_{j-1})}{-\,b_3} + \frac{b_1\, exp(-\,(b_2 + b_3)z_{j-1})}{b_2 + b_3}\right\}$$

Equation 9.32

$$SNG = (b_0 - b_1 exp(-\,b_2 z_m))\,(exp(-b_3 z_m))\,[3.51\,Log_{10}CBR - 0.85(Log_{10}CBR)^2 - 1.43]$$

Equation 9.33

SNP = Adjusted structural number of the pavement.
SNA = Contribution of surfacing and base layers.
SNS = Contribution of the sub-base and selected fill layers.
SNG = Contribution of the subgrade.
n = Number of base and surfacing layers (i = 1, n).
a_i = Layer coefficient for base or surfacing layer i.
h_i = Thickness of base or surfacing layer i, in mm.
m = Number of sub-base and selected fill layers (j = 1, m).
a_j = Layer coefficient for sub-base or selected fill layer j for season s.
z = Depth parameter measured from the top of the sub-base (underside of base), mm.
z_j = Depth to the underside of the j^{th} layer ($z_0 = 0$), in mm.
CBR = In situ subgrade CBR

The values of the model coefficients b_0 to b_3 are given in Table 9.11.

Table 9.11 Adjusted Structural Number Model Coefficients

b_0	b_1	b_2	b_3
1.6	0.6	0.008	0.00207

9.7.1.3 Determination of Effective Structural Number (SN_{eff}) of Existing Pavement from Deflections

The method is used to determine the structural number to be used in the design of the overlay that is representative of the existing pavement.

There are 2 methods of determining the Effective Structural Number:

a. Graphical method – in this method the adjusted structural number (SNP) is plotted against the central deflections as illustrated in the example given in Figure 9.6.

i. Plot the mean line i.e., the line of best fit.

ii. Write the equation of the line of best fit and the R2 value.

iii. Calculate the 90% confidence SNP values from the values on the mean line using Equation 9.34. For 80% confidence if required, use Equation 9.35.

$$90\%\ Confidence = mean - 1.3S$$

Equation 9.34

$$80\%\ Confidence = mean - 0.842S$$

Equation 9.35

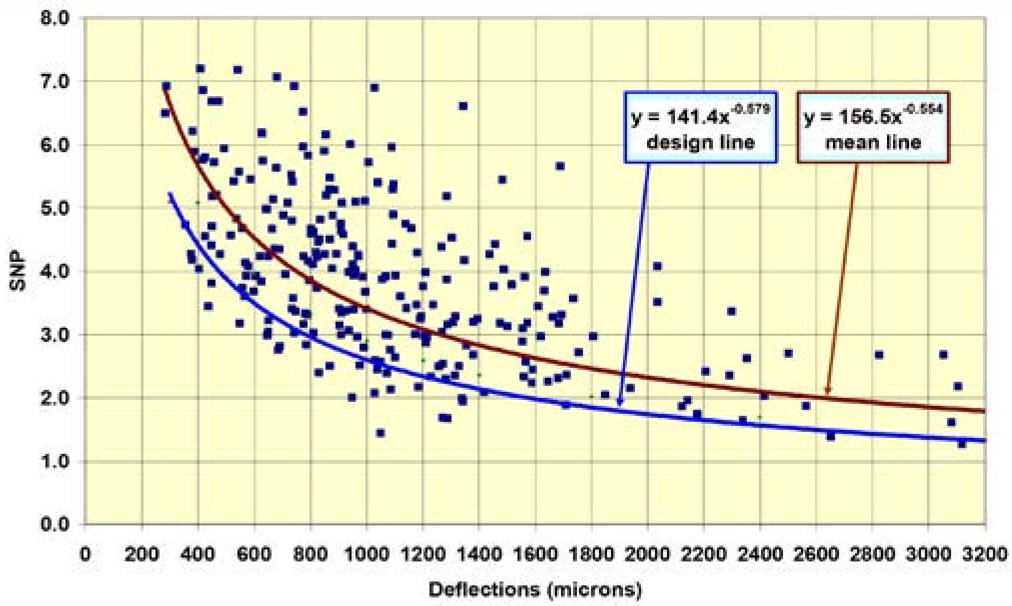
iv. Plot the values against deflections, d_0.

v. Apply a line of best fit for 90% confidence values of SNP.

vi. Write down the equation of SNP_{90} and d_{90}.

vii. SNP_{90} is the effective structural number, SN_{eff}.

Figure 9.6 Example of a Plot of SNP vs Deflections



b. Determination of SNeff using the effective modulus of pavement. This method assumes that the pavement structural capacity is a function of the total thickness and overall stiffness as stipulated in Equation 9.36 below.

$$SN_{eff} = \frac{6T_t}{100} x3\sqrt{E_p}$$

Equation 9.36

Where,

T_t = Total thickness of pavement (cm) i.e., of all layers above the subgrade

E_p = Effective modulus of the pavement layers above the subgrade, kPa.

To determine E_p, obtain the following parameters from the non-destructive deflection tests.

i. D_0 = central deflection adjusted to 20°C.

ii. P = applied load, kg.

iii. D = total thickness of pavement layers above the subgrade, cm.

iv. D_r = deflection at distance r from the centre of the load, cm.

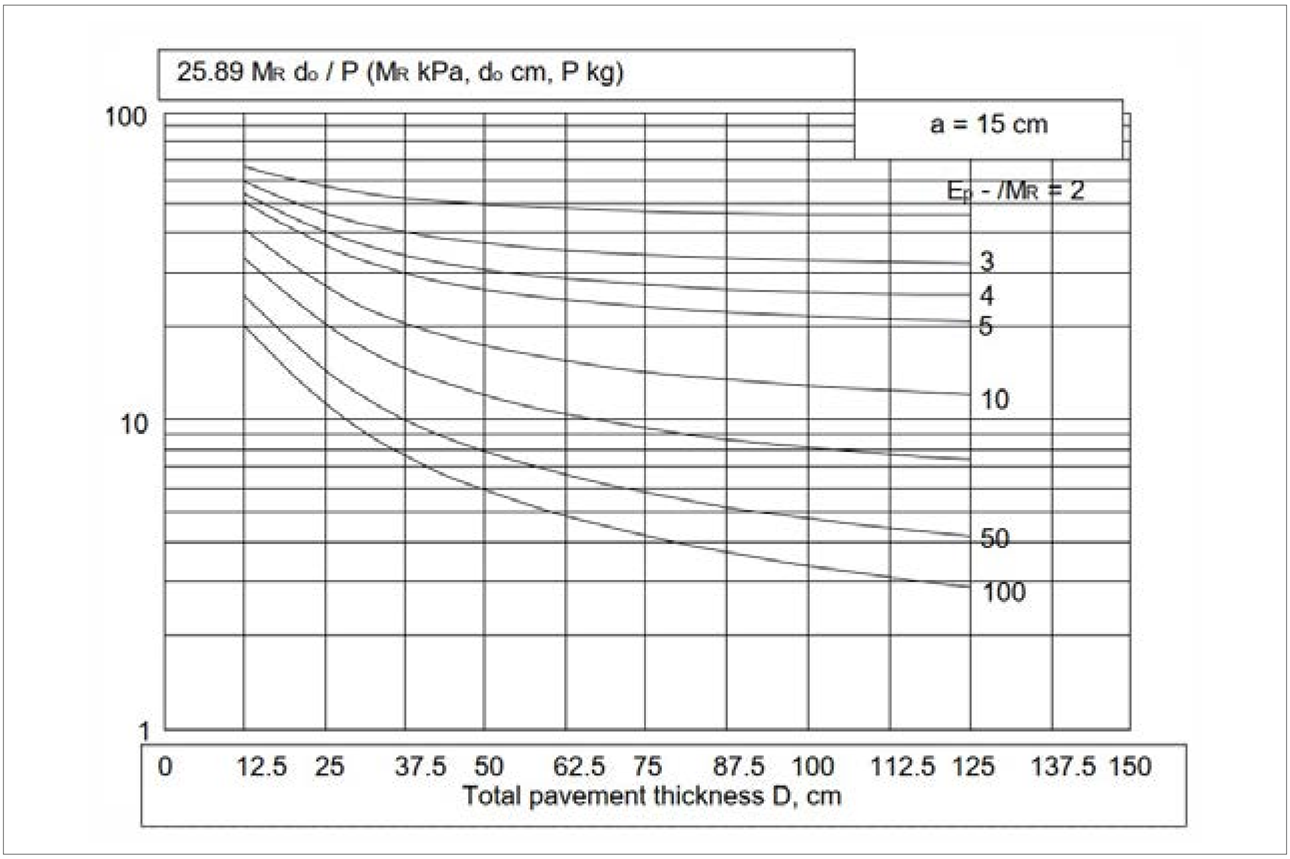
v. R = distance from the centre of the load, cm.

vi. A = load plate radius, cm.

vii. Using Figure 9.7, calculate the ratio E_p/M_R, where M_R is the subgrade resilience, kPa. The plot shows $M_R D_0/P$ vs Total pavement thickness. Read the corresponding value of E_p/M_R. Using subgrade M_R value from back calculation or estimation from d_{1200}, d_{1500} and d_{1800} (Section 6.3.4) calculate E_p.

1
2
3
4
5
6
7
8
9
Structural Design of Overlays for Flexible Pavements

Figure 9.7 Determination of E_p/M_R



The overlay design mitigates the structural deficit by overlaying with the appropriate layer(s) on top of the existing pavement to increase the traffic load-carrying capacity of the pavement.

9.7.2 Design Process

The design process involves:

1. Characterisation of the pavement and its constituent layers, see Section 6.3.3 (detailing parameters required for rehabilitation design.
2. Determination of design traffic, Chapter 5.

Determination of layer strength coefficients, from material parameters. The layer strength coefficients for different pavement layers and their constituent materials are given in Table 9.12. Experience and engineering judgement are required while assigning appropriate values of layer strength coefficients, especially for deteriorated or deteriorating layers like AC and DBM or cracked cement treated base (CTB). The material strength parameters determining the layer strength coefficients are measured in the laboratory, and common in-situ tests like DCP should be carried out. E-moduli are determined in the laboratory for new materials and through samples collected during field investigations or non-destructive tests like FWD deflections. Layer strength coefficients are determined from Table 9.12.

Table 9.12 Pavement Layer Strength Coefficients

Layer	Layer Type	Condition	Coefficient
Surfacing	Surface dressing		a_i = 0.1
	New asphalt concrete a,b,c wearing	MR_{30} = 1500 MPa	a_i = 0.30
		MR_{30} = 2000 MPa	a_i = 0.35
		MR_{30} = 2500 MPa	a_i = 0.40
		$MR_{30} \geq$ 3000 MPa	a_i = 0.45
Road base	Asphalt concrete	As above	As above
	Granular unbound	Default	$a_i = (29.14\ CBR - 0.1977\ CBR^2 + 0.00045\ CBR^3)\ 10^{-4}$
		GB 1 (CBR > 100%)	0.145
		GB 2 (CBR = 100%)	0.14
		GB 3 (CBR = 80%) With a stabilised layer underneath	0.135
		With an unbound granular layer underneath	0.13
		GB 4 (CBR = 65%)[(d)]	0.12
		GB 5 (CBR = 55%)[(d)]	0.107
		GB 6 (CBR = 45%)[(d)]	0.1
	Bitumen-treated gravel and sands	Marshall stability = 2.5 MN	a = 0.135
		Marshall stability = 5.0 MN	a = 0.185
		Marshall stability = 7.5 MN	a = 0.23
	Cemented	Equation	$a_i = 0.075 + 0.039\ UCS - 0.00088(UCS)^2$ for values given in ITS: UCS – 48.43(ITS)
		CB1 (UCS = 6.0-9.0)	a = 0.28
		CB 2 (UCS = 3.0 – 6.0 MPa)	a = 0.18
		CB 3 (UCS = 1.5 – 3.0 MPa)	a = 0.13
Sub-base	Granular unbound	Equation	$a_j = 0.075 + 0.184(log_{10}\ CBR) - 0.0444(log_{10}\ CBR)^2$
		GS (CBR = 30%)	a = 0.105
		GC (CBR = 15%)	a = 0.08
	Cemented	CB 3 (UCS = 0.7 – 1.5 MPa)	a = 0.1

Notes:
a) *See the discussion above.*
b) *Unconfined Compressive Strength (UCS) is quoted in MPa at 14 days.*
c) *MR30 is the resilient modulus by the indirect tensile test at 30 °C.*
d) *Used for low-volume roads (see LVR manual)*

The layer strength coefficients for the different materials that form the subgrade or foundation, the pavement and the surfacing are derived from Table 9.12 and provided in Table 9.13 below.

Table 9.13 Layer Strength Coefficients for Pavement Materials

Code	Material	Layer Strength Coeffi-cient (a)
G3	Clayey and silty sands of minimum 4 days soaked CBR of 3%	-
G8	Clayey and silty sands, natural gravels or natural materials blended with up to 20% stone aggregates of minimum 4 days soaked CBR of 8%	-
G10	Clayey and silty sands, natural gravels or natural materials blended with up to 30% stone aggregates of minimum 4 days soaked CBR of 10%	-
G14	Clayey and silty sands, natural gravels or natural materials blended with up to 30% stone aggregates of minimum 4 days soaked CBR of 14%	0.08
G20	Clayey and silty sands, natural gravels or natural materials blended with up to 30% stone aggregates of minimum 4 days soaked CBR of 20%	0.08
G23	Clayey and silty sands, natural gravels or natural materials blended with up to 30% stone aggregates of minimum 4 days soaked CBR of 23%.	0.08
G25	Clayey and silty sands, natural gravels or natural materials blended with up to 30% stone aggregates of minimum 4 days soaked CBR of 25%	0.08
G30	Clayey and silty sands, natural gravels or natural materials blended with up to 30% stone aggregates of minimum 4 days soaked CBR of 30%	0.105
G45	Clayey and silty sands, natural gravels or natural materials blended with up to 30% stone aggregates of minimum 4 days soaked CBR of 45%	0.105
G50	Natural gravels or natural materials blended with up to 30% stone aggregates of minimum 4 days soaked CBR of 50%; OR crushed stone gravel, crusher run of indeterminate CBR but complying with the gradation	0.105
G80	Natural gravels or natural materials blended with up to 30% stone aggregates of minimum 4 days soaked CBR of 80%; OR crushed stone gravel, crusher run of indeterminate CBR but complying with the gradation	0.135
GCS-F	Crushed stone aggregates Class F of minimum 4 days soaked CBR 30%. Mostly weathered rock.	0.1
GCS-E	Crushed stone aggregates Class E of minimum 4 days soaked CBR of 50%, maximum ACV 35% and LAA 50%. Mostly soft stone, and partially weathered rock.	0.107
GCS-D	Crushed stone aggregates Class D of minimum 4 days soaked CBR of 80%, maximum ACV 35% and LAA 50%. Mostly partially weathered granites, basalts, and other rocks.	0.13
GCS-C	GCS Class C of maximum ACV 30% and LAA 40%. Mostly fresh corals, and metamorphic rocks.	0.14
GCS-B	GCS Class B of maximum ACV 28% and LAA 35%. Mostly fresh granites, basalts, and other igneous rocks.	0.14
GCS-A	GCS Class A of maximum ACV 25% and LAA 30%. Mostly fresh granites, basalts, and other igneous rocks.	0.145
HPS	Hand Packed Stone of maximum ACV 35% and LAA 50%. Mostly fresh trachytes, soft stone, basalts, and other igneous rocks.	0.14
MAC	Dry-bound and wet-bound macadam. Complies with parent rock requirements for GCS grades	0.30
TEL	Telford bases	0.40
HIG50	Lime and hydraulically improved granular material of minimum CBR of 50% after 7 days cure & 7 days soak.	0.107
HIG60	Lime and hydraulically improved granular material of minimum CBR of 60% after 7 days cure & 7 days soak.	0.12
HIG100	Lime and hydraulically improved granular materials of minimum CBR of 100% after 7 days cure & 7 days soak.	0.14
HIG160	Lime and hydraulically improved granular materials of minimum CBR of 160% and UCS 1.0 to 2.0 MPa after 7 days cure & 7 days soak	0.145

Code	Material	Layer Strength Coeffi-cient (a)
HMS1	Hydraulically modified stone of minimum UCS 1.2 MPa and maximum UCS 2.5 MPa after 7-day cure & 7-day soak	0.13
HBS3	Hydraulically bound stone of minimum UCS 3.0 MPa after 7-day cure & 7-day soak	0.18
HBS6	Hydraulically bound stone of minimum UCS 6.0 MPa after 7-day cure & 7-day soak	0.27
HBS9	Hydraulically bound stone of minimum UCS 9.0 MPa after 7-day cure & 7-day soak	0.35
BSM50	Bitumen Stabilised Material of minimum soaked ITS of 50 kPa. (2.4 MPa)	0.16
BSM100	Bitumen Stabilised Material of minimum soaked ITS of 100 kPa. (4.8 MPa)	0.24
BSM175	Bitumen Stabilised Material of minimum soaked ITS of 175 kPa. (8.5Mpa)	0.34
DBM	Dense Bitumen Macadam of minimum Marshall Stability 9 kN and Modulus 5000 MPa.	0.45
EME	EME Asphalt of minimum modulus 8000 MPa.	0.45
SBMa	Sand bitumen mix (silty clayey sand) of minimum Marshall Stability 3.75 kN.	0.3
SBMb	Sand bitumen mix (clean sand) of minimum Marshall Stability 2.5 kN.	0.2
DSD	Surface dressing made with single sized aggregates	0.1
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.	0.1
CMA	Cold mix asphalt made with emulsion and graded stone 0/10 or 0/14	0.3
OTA	Otta seal made with graded aggregate and soft penetration bitumen or cut-back or emulsion	0.1
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 least two layers). Split application encouraged.	0.1
SAN	Sand bitumen mix of minimum Marshall Stability 3 kN and Modulus 1000 MPa.	0.2
GAP	Gap graded asphalt of minimum Marshall Stability 3 kN and Modulus 1500 MPa.	0.3
ACII	Flexible asphalt of minimum Marshall Stability 6 kN and Modulus 2500 MPa.	0.4
ACI	High stability asphalt concrete of minimum Marshall Stability 9 kN and Modulus 4000 MPa.	0.45
ACIb	High stability asphalt concrete (for binder course) of minimum Marshall Stability 9 kN and Modulus 4000 MPa.	0.45
SMA	Stone mastic asphalt of minimum Marshall Stability 9 kN and Modulus 5000 MPa.	0.45
ICB	Interlocking cobblestone paving of minimum UCS 25 MPa	0.3
IPB	Interlocking concrete paving blocks of minimum UCS 25 MPa	0.3
CP-1	Concrete for TC1 and lower	-
CP-2	Concrete for jointed unreinforced concrete (JUC), jointed reinforced concrete (JRC), continuously reinforced concrete pavement (CRCP)	-
CP-3	Concrete for continuously reinforced concrete base (CRCB)	-
CP-4	Concrete for roller compacted concrete (RCC)	-

3. Determination of target/design structural number (SN_d) – this is the structural number that is required to carry future traffic loading (CESA). There are 2 approaches to this.
   a. Target SN_d from catalogues for pavement types in RDM 3.4 based on Foundation Classes
   b. Determination of SN_d from catalogue-based subgrade (CBR) classes.

9.7.2.1 Determination of SNd using Catalogues in RDM 3.4 Based on Foundation Classes

This approach involves the determination of the Standard Pavement Types, provided in RDM 3.4. An example of the catalogues is given in Appendix B showing the Standard Pavement Type 1.

Target SN_d is calculated from the layer strength coefficients given in Table 9.13 and layer thickness given in the catalogues in RDM 3.4 using the structural number equations given in Section 9.7.1. These would be the calculated target SN_d values for the different types of pavements for the given foundation class, pavement materials used and forecasted future traffic loading or design traffic loading class.

9.7.2.2 Determination of Target SNd using Catalogues Based on Subgrade Classes.

Target SN_d from the catalogues based on subgrade class is given in Table 9.30.

Table 9.14 Target SN_d for Different Pavement Structures Based on Subgrade Classes

Chart No.	Subgrade	TC1	TC1.5	TC3	TC6	TC10	TC17	TC30	TC50	TC80
		1.0	1.5	3.0	6.0	10.0	17.0	30.0	50.0	80.0
Chart A1	S1	2.86	2.99	3.30	3.47	3.66				
	S2	2.44	2.57	2.80	3.05	3.28				
	S3	1.95	2.08	2.31	2.50	2.74	Includes 0.1 for SD			
	S4	1.50	1.63	1.87	2.07	2.31				
	S5	1.17	1.30	1.43	1.56	1.76				
	S6	0.88	1.01	1.01	1.15	1.28				
Chart A2	S1	2.82	2.95	3.25	3.45	3.61	3.82			
	S2	2.40	2.53	2.82	3.04	3.23	3.43			
	S3	1.96	2.09	2.31	2.57	2.78	2.91			
	S4	1.65	1.78	1.91	2.16	2.31	2.44			
	ss	1.26	1.40	1.52	1.78	1.91	2.04			
	S6	0.88	1.01	1.15	1.28	1.52	1.65			
Chart A3	S1	2.74	2.87	3.00	3.20	3.31	3.44			
	S2	2.39	2.51	2.64	2.85	3.06	3.18			
	S3	2.02	2.15	2.25	2.47	2.67	2.80			
	S4	1.59	1.72	1.95	2.19	2.39	2.52			
	S5	1.28	1.41	1.61	1.85	1.95	2.08			
	S6	0.87	0.99	1.12	1.25	1.38	1.51			
Chart B	S1		3.04	3.25	3.45	3.68				
	S2		2.58	2.79	3.00	3.23				
	S3		2.08	2.29	2.49	2.72				
	S4		1. 77	1.87	2.08	2.31				
	S5		1.44	1.54	1.64	1.87				
	S6		1.02	1.15	1.28	1.41				

Chart No.	Subgrade	TC1	TC1.5	TC3	TC6	TC10	TC17	TC30	TC50
		1.0	1.5	3.0	6.0	10.0	17.0	30.0	50.0
Chart C1	S1					4.61	5.06	5.61	
	S2					4.08	4.53	5.08	
	S3					3.59	4.04	4.49	
	S4					3.16	3.61	4.06	
	S5					2.79	3.27	3.75	
	S6					2.44	2.92	3.40	
Chart C2	S1					4.44	5.04	5.30	
	S2					4.04	4.51	4.82	
	S3					3.51	3.99	4.33	
	S4					3.20	3.67	4.02	
	S5					2.94	3.29	3.63	
	S6					2.68	3.02	3.37	

Chart No.	Subgrade	TC1	TC1.5	TC3	TC6	TC10	TC17	TC30	TC50	TC80
		1.0	1.5	3.0	6.0	10.0	17.0	30.0	50.0	80.0
Chart D	S1					4.66	5.00	5.34	5.58	5.93
	S2					4.31	4.55	4.89	5.12	5.48
	S3					3.93	4.16	4.50	4.74	5.09
	S4					3.50	3.73	4.07	4.31	4.66
	S5					3.19	3.42	3.66	3.90	4.25
	S6					2.78	3.01	3.25	3.48	3.84

c. Alternatively, the AASHTO Equation 9.37 below can be resolved for SN, which would be the design structural number (SN_d):

$$Log_{10}(W_{80}) = Z_R x S_p + 9.36 x Log_{10}(SN+1) - 0.20 + \frac{Log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.40 + \left[\frac{1094}{(SN+1)^{5.19}}\right]} + 2.32 x Log_{10}(M_R) - 8.07$$

Equation 9.37

Where,

W_{80} = Design traffic loading in ESAs.

Z_R = Standard normal deviate.

S_0 = Combined standard error of the traffic prediction and performance prediction.

SN = Structural Number (an index of the total pavement thickness required).

$= a_1h_1 + a_2h_2m_2 + \ldots\ldots a_ih_im_i$

ΔPSI = Difference between the initial design serviceability index P_o and the design terminal serviceability index P_t.

M_R = subgrade resilient modulus in psi. (Conversion: psi = kPa/6.895)

Values of Z_R are given in Table 9.15.

Table 9.15 Determination of Z_R (Standard Normal Deviate)

Reliability (%)	ZR	Reliability (%)	ZR
50	-0.000	93	-1.476
60	-0.253	94	-1.555
70	-0.524	95	-1.645
75	-0.674	96	-1.751
80	-0.841	97	-1.881
85	-1.037	98	-2.054
90	-1.282	99	-2.327
91	-1.340	99.9	-3.090
92	-1.405	99.9	-2.750

Values of S_0 are given in Table 9.16.

Table 9.16 Standard Deviations

Source	Flexible pavements	Rigid pavements
AASHTO Road Test S_n	0.35	0.25
AASHTO Road Test S_0	0.45	0.35
AASHTO Road Test S_0	0.40 - 050	0.35 – 0.40

9.7.3 Determination of the Deficit in Structural Number, $SN_{deficit}$

In rehabilitation design, a comparison is made between the existing pavement Adjusted Structural Number (SNP) and the design structural number (SN_d) required for the pavement to carry future traffic loading through the designated design life. The purpose of the rehabilitation design process is to determine SNP or SN_{eff} and SN_d, and calculate the deficit using Equation 9.38 and Equation 9.39:

$$SNP_{deficit} = SN_d - SNP$$

Similarly:

$$SNP_{deficit} = SN_d - SN_{eff}$$

Equation 9.38

Overlay thickness calculation, (h_0):

$$h_0 = 25.4\left(\frac{SN_d - SNP}{a_1}\right)$$

Similarly:

$$h_0 = 25.4\left(\frac{SN_d - SN_{eff}}{a_1}\right)$$

Equation 9.39

Where,

a_1 = layer strength coefficient for the overlay (for AC it would generally be 0.35)

h_0 = thickness of the overlay

Where the structural deficiency is zero or negative, no overlay is required, and only appropriate maintenance interventions are required.

9.7.4 Selection of Overlay Options

At this stage, the structural deficit is known and the overlay options can be determined. Refer to Section 8.4 for overlay options, their characteristics, and suitability for the existing pavement and traffic.

Overlays are applied on existing road pavements to strengthen them:

1. The most common choice is AC Type 1 or Type 2. AC has a high layer strength coefficient to strengthen and provide an impermeable wearing surface, comfort and good aesthetics for road users.
2. The second option is GCS with DSD or AC wearing course. GCS provides extra strength and more importantly, acts as a crack control mechanism. Reflective cracking is usually a big problem in the rehabilitation of pavements and the root cause of premature failures.
3. Cement-stabilised bases with bituminous surfaces are also high on the list of choices and work for low and high traffic. The problem is that high cement content is required for fine bases. For example, sand bases require as high as 5% to 7% cement content by mass. As a result, shrinkage cracking is inevitable though it can be minimised through adequate curing during construction.

The choice of overlays is also greatly influenced by:

1. The availability of materials and the life cycle costs. In coastal areas, cement stabilised sand bases are more common and so is ETB. Where stone quarries are available, GCS is the most used.
2. The choice can also be influenced by functionality. For example, macadam and Telford bases are for longevity and robustness in adverse conditions like extreme flooding. They are appropriate choices for climate resilience.

These are some of the considerations the design engineer should make.

9.7.5 Rationalising Overlay Thickness over the Uniform Sections

The structural deficiency is determined point by point. It is not appropriate to use averages at this stage.

Produce a plot of the structural deficit at each chainage as shown in the example given in Figure 9.8.

For constructability, overlay thickness cannot be varied over short distances. The overlay thicknesses should be rationalised over each uniform section. An example is given in Figure 9.9 and it applies to both LVRs and HVRs. The short sections or localised areas that show requirements for significantly thicker overlays i.e., well above the overlay thickness line will need significant repairs before the overlay is placed. This might involve the removal of damaged pavement layers and replacing them with new materials. This will help to save on construction costs because the repaired localised sections will be taken out of the analysis thus reducing the overlay thickness required.

Figure 9.8 Example of a Plot of Structural Deficiency

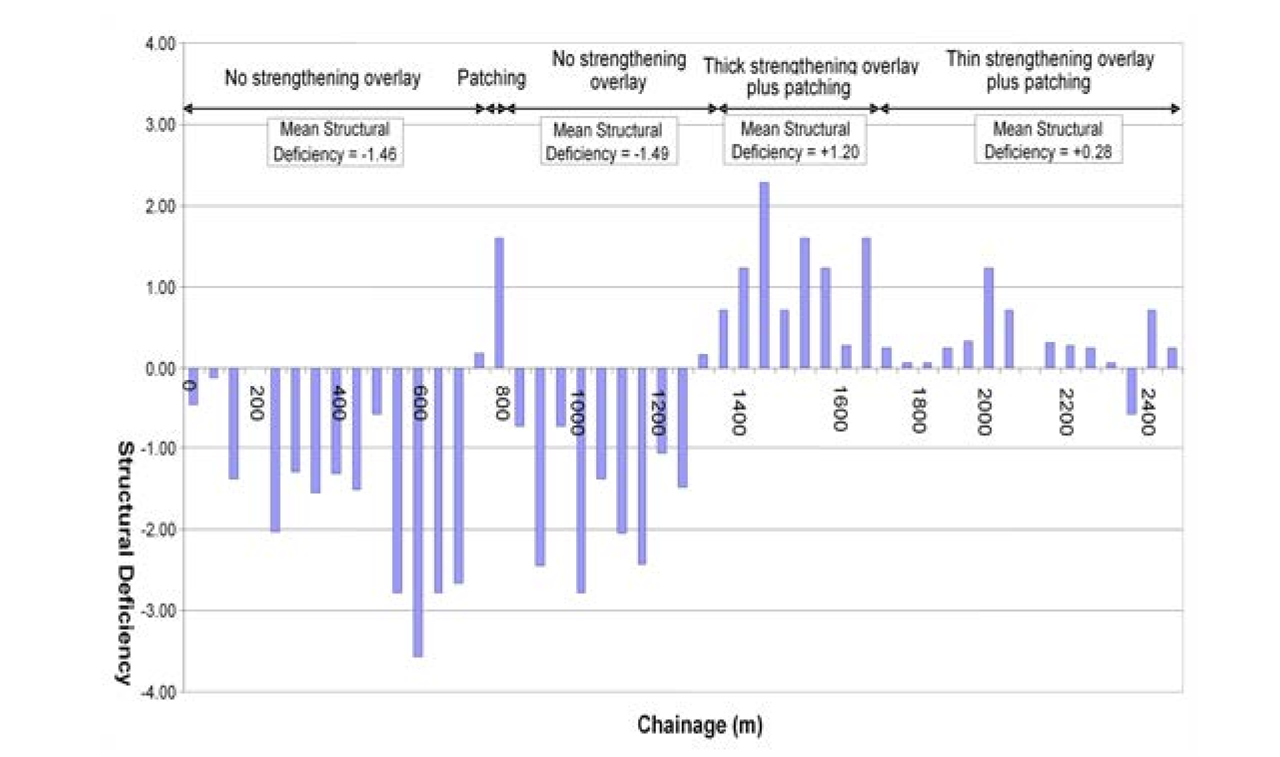
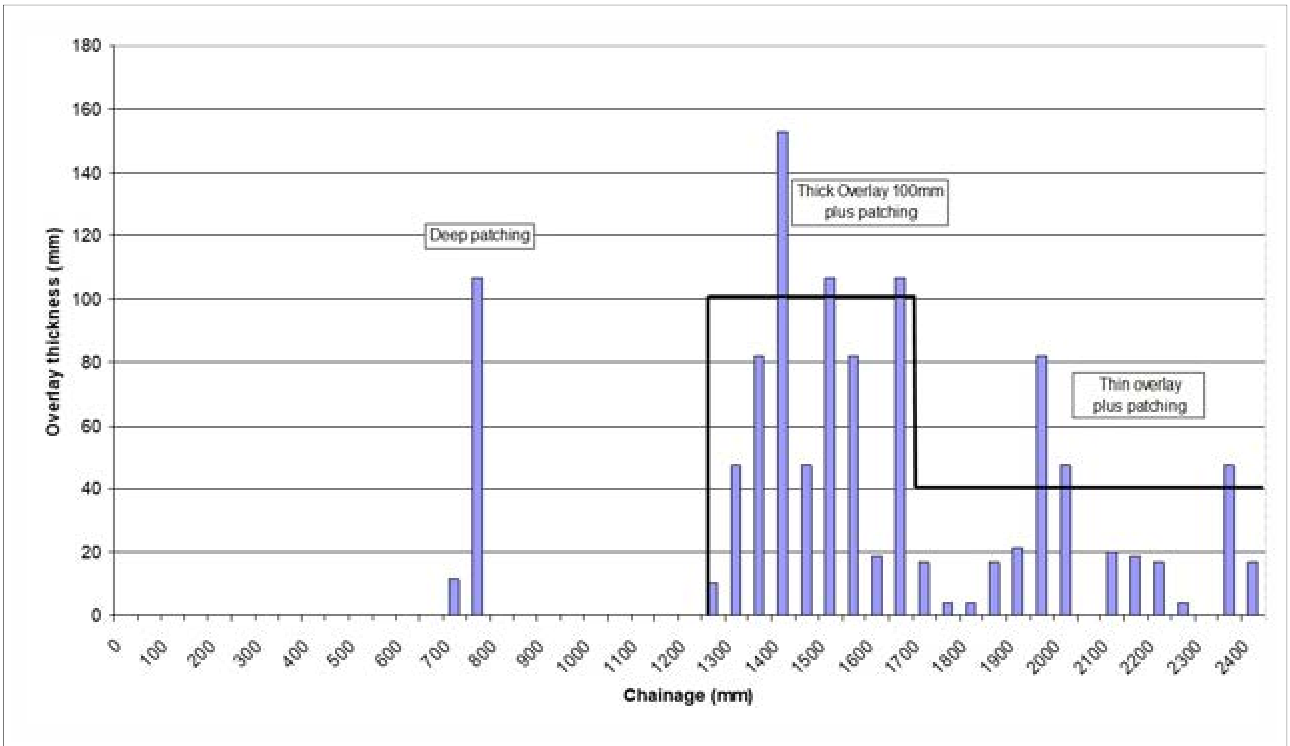


Figure 9.9 Rationalising Overlay Thickness Over Each Uniform Section



The design overlay thicknesses are determined using the reliability given in Table 9.17. This can be determined from the overlay.

Table 9.17 Percentage Reliability to be used for Overlay Design

Item No.	Road Standard	Reliability (%)
1.	High volume roads	90
2.	Low volume roads	80

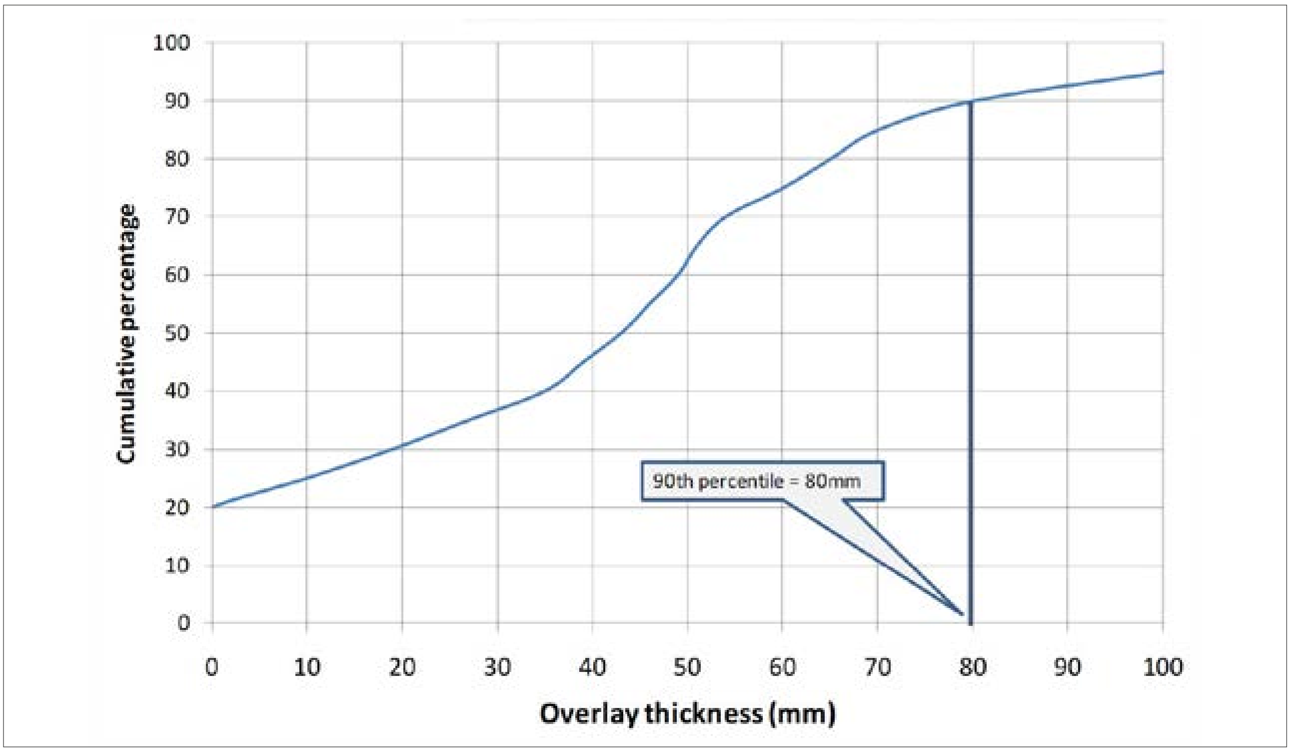
The following method can be used to determine the design thicknesses of overlays based on the percentage reliability given in Table 9.17. A simple method of cumulative sums (CUMSUM) can be used to determine the percentile values. Determine percentages of the number of values in a range of overlay design thicknesses as illustrated in Table 9.18. The ranges can be varied depending on the distribution of the data.

Plot cumulative percentages against averages of the values in the ranges to obtain a cumulative frequency distribution curve, Figure 9.11. Read the overlay thickness corresponding to the percentage reliability designated for the road standard.

Table 9.18 Example for the Calculation of Cumulative Percentages

Chainages	Ranges of overlay thicknesses	Averages	Number of values in the range	Percentages (%)	Cumulative Percentages (%)
	0 – 20	15			
	30 - 40	35			
	40 - 50	45			
	50 – 60	55			
	70 - 80	75			
	80 - 90	85			
	90 - 100	95			100
Total				100	

Figure 9.10 Example of Frequency Distribution Curve for Determination of Design Overlay Thickness for 80% Reliability



Structural Design of Overlays for Flexible Pavements

9.8 Performance Method

The performance method, also called the deflection reduction method is generally developed from historical deflection data. The method involves the development of the relationship between deflections and rut depths. The science behind it is that both rutting and deflections increase with cumulative traffic loading.

Also, rutting is generally used as one of the key pavement failure criteria. Jurisdictions like South Africa, Australia, the UK and Kenya use 20 mm rut depth for high-volume roads and in some cases 30 mm rut depth for low-volume roads to define failure conditions. The rut depth failure criterion is referred to as the critical rut depth.

9.8.1 Procedure for Deflection Reduction Method

In the deflection reduction method, the overlay design thickness is determined as follows:

1. Using the plot of rut depth and deflections determine the critical deflection corresponding to the critical rut depth given as the failure criterion for the pavement.
   a. Measure rut depth and deflections at the same test points.
   b. Plot rut depth against deflection, Figure 9.11.
   c. Determine the cumulative traffic volume at the time of testing.
   d. Draw a line of best fit.
   e. Convert the values on this line using Equation 9.40 for 90% confidence:

$$D_{90} = D_{mean} + 1.3\,SD$$

Equation 9.40

Where,

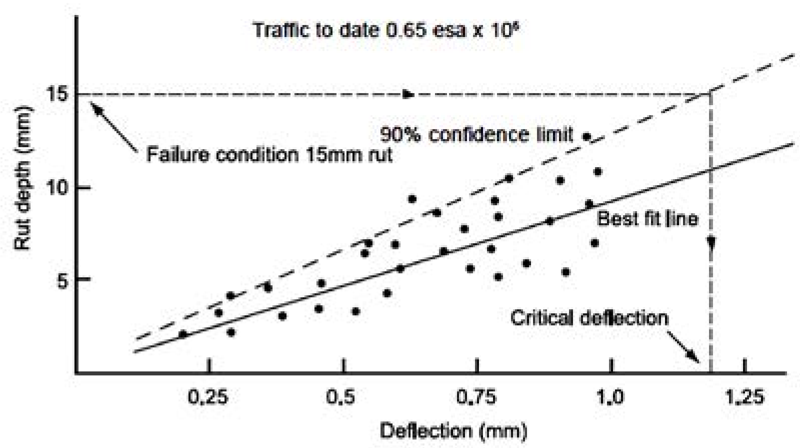
D_{90} = 90% reliability/confidence

D_{mean} = mean deflection

SD = standard deviation

   f. Plot rut depth against D_{90} to get the 90% confidence line.
   g. Using the rut depth failure criterion or critical rut depth given in the standards, draw a line to the 90th percentile line and read the corresponding critical deflection.

Figure 9.11 Example for Determination of Critical Deflection With CRD of 15 mm

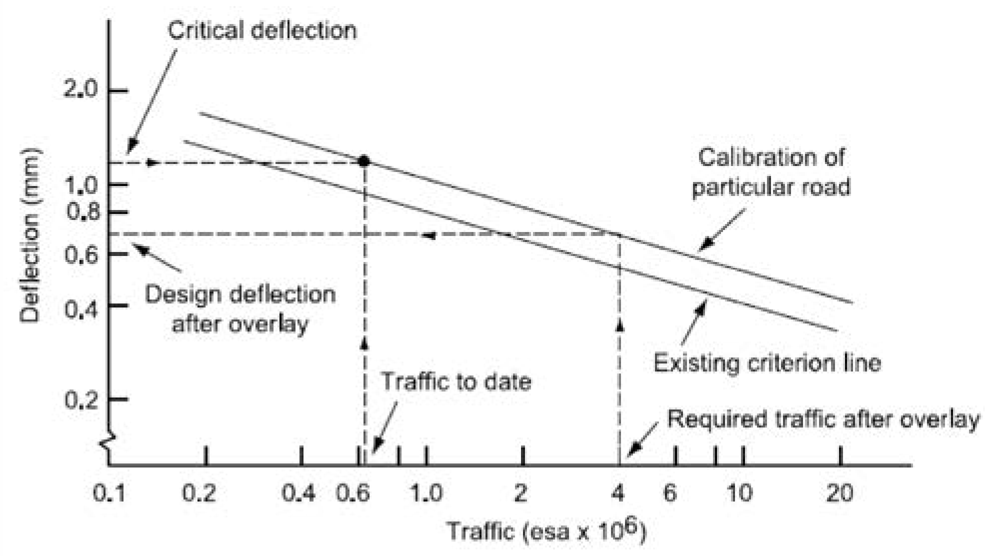


9.8.2 Determination of Design Deflection Using the Deflection Reduction Method

Using historical deflections and traffic loading for the road and if possible, from similar roads nearby:

1. Plot a graph of initial deflection from a new road (if the road has not been overlaid before) or the last overlay against cumulative traffic loading at the point of failure for several roads in the region or zone of uniform climatic conditions to produce a trend line, Figure 9.13. The trend line is the existing criterion line. The point of failure or failure criteria is the critical rut depth at which the pavement requires an overlay, see Section 7.2 i.e. rut depth of 15 mm for minor overlays, 20 mm for major overlays on high-volume roads and 25 mm for major overlays on low volume roads.
2. Plot a point on the graph corresponding to the critical deflection and the cumulative traffic loading to date for the road, draw a line through the point and parallel to the existing trend line. This would be the calibration of the road under design or the calibration line.
3. Determine the future cumulative traffic loading (Chapter 5), draw a line to the calibration line and read off the corresponding deflection, which is the design deflection after the overlay.

Figure 9.12 A Plot of Historical Deflection and Cumulative Traffic Loading



9.8.3 Calculation of Overlay Thickness Using the Deflection Reduction Method

Calculate the overlay thickness using Equation 9.41:

$$T = \frac{0.036 + 0.818\,D_r - D_d}{0.0027D_r}$$

Equation 9.41

Where,

D_r = Representative deflection, mm.

D_d = Design deflection, mm.

T = Overlay thickness, mm.

This relationship is valid for:

a. Axle load of 62.3 kN, hence all deflections carried out at different axle loads should be normalised to 62.3 kN.

b. Representative deflections shall be between 0.25 mm and 1.25 mm, and overlay thickness of 40 mm to 150 mm.

10 Structural Design of Overlays for Rigid and Semi-rigid Pavement

10.1 General

The section covers overlay designs, such that the final pavement is considered rigid or semi-rigid, e.g. a concrete overlay on an existing asphalt/composite/concrete pavement.

For asphalt overlays on an existing asphalt/composite/concrete pavement see Chapter 9. The design of new rigid pavements is provided in RDM 3.5.

Concrete overlays are usually carried out for structural reasons as a rehabilitation option to strengthen an existing road. The choice of concrete overlay will depend upon the type and condition of the existing pavement.

There are many different methods of concrete overlay design. The following design methods are covered in this Chapter:

1. Section 10.6: UTRCP (Ultra-Thin Reinforced Concrete Pavement) where a fixed 50mm thick lightly reinforced concrete overlay is applied to extend the life of low-volume urban or rural roads (0.1 MCESA to 3.0 MCESA). This is effectively a non-structural overlay.
2. Section 10.7: Empirical Concrete Overlay Design Method
3. Section 10.8: Modified Empirical Concrete Overlay Design Method
4. Section 10.9: Structural Deficit Method (based on UK/AASHTO design principles)
5. Section 10.10: Mechanistic-Empirical Design Method (based on the India/AASHTO Method)

10.1.1 Types of Concrete Pavements

There are several types of rigid pavements given in RDM 3.5. Their behaviour and deterioration differ significantly. Table 10.1 provides the different types of rigid pavements. Details of the reinforcement general layout are given in RDM 3.5.

Table 10.1 Main Types of Rigid Pavement – Advantages and Disadvantages

No.	Pavement Type	Traffic Level Suitability	Cost	Advantages (Pros)	Disadvantages (Cons)
1	Cobblestone Paving (CP)	Low (0.1 MCESA to 1 MCESA) and speeds <50 km/h.	$	• Basic (low-cost) form of pavement. • Often local material is available. • Easily laid with minimum plant and unskilled labour. • Dry construction - can be trafficked immediately. • Can be dug up and re-laid if required.	• Poor ride quality& skid resistance • Unsuitable for high-speed roads • Noise & rutting may be an issue.
2	Block Paving (BP)	Low (0.1 MCESA to 1 MCESA) and speeds <50 km/h.	$	• Basic (low-cost) form of pavement. • Durable, factory-made blocks. Range of shapes, colours • Easily laid with minimum plant and unskilled labour. • Dry construction - can be trafficked immediately. • Can be dug up and re-laid if required.	• Unsuitable for high-speed roads • Poor ride quality& skid resistance • Noise & rutting may be an issue. • Can need frequent maintenance if high traffic.
3	Jointed Unreinforced Concrete pavement (JUCP). (Unreinforced, square joints with dowels).	Low/Medium/ High (0.1 MCESA to 50 MCESA)	$$	• Basic (low-cost) form of concrete pavement. • Less steel is required than CRCP. • Better performance than undowelled JUCP. • Good ride quality if paver-laid. • Durable pavement, if timely repairs.	• Joints are the main weakness and can be a source of problems throughout life. • Recurring maintenance required. • Less suitable for high temp range areas • Concrete surface - noise may be an issue.

No.	Pavement Type	Traffic Level Suitability	Cost	Advantages (Pros)	Disadvantages (Cons)
4	Jointed Unreinforced Concrete Pavement (JUCP). (Unreinforced, skew joints without dowels).	Low traffic routes such as rural roads (< 1 M CESA)	$	• Lowest cost form of concrete pavement. • Can be hand-laid with minimum plant. • Less steel is required than CRCP. • Good ride quality if paver-laid.	• Only suitable for low-volume roads. • Joints will cause problems throughout life. • Skew joints prone to corner cracks. • Poor load transfer -HGVs will damage joints. • Concrete surface - noise may be an issue.
5	Jointed Reinforced Concrete Pavement (JRC).	Medium/High (1 MCESA to 50 MCESA)	$$$$	• > 80% fewer joints than JUCP (joints at 25 m rather than 4.5 m), so fewer joint problems. • Reinforced, so fewer cracking issues than JUCP. • Less joint movement and end movement. • Good ride quality if paver-laid. • Better than JUCP if subgrade settlement issues.	• Joints are a weakness and can cause problems. • Recurring maintenance required. • Concrete surface, so noise may be an issue.
6	Roller Compacted Concrete Pavement (RCC). (For design guide see references) (With >90mm asphalt).	Medium/High/ Very High (1 MCESA to 600 MCESA) (See cons re surfacing)	$$$	• Quick/easy/cheap to construct. No steel/formwork. • Can be used by traffic/overlaid soon after paving. • Asphalt surfacing so low noise, etc. • Lower cement content so less shrinkage cracking. • Lower maintenance costs than asphalt over a lifetime. • Can be used in areas with poor subgrades.	• Likely poor ride quality/skid resistance. • Higher-speed roads (> 60 kph) will need an asphalt surfacing for good skid resistance and surface evenness.
7	Continuously Reinforced Concrete Base (CRCB). (with minimum 100mm thick asphalt surfacing).	High/Very high (10 MCESA to 400 MCESA). Particularly for very high traffic levels (>50 MCESA).	$$$$$	• Combines concrete strength & quiet asphalt surface. • Excellent durability. No problematic joints. • Suitable for very heavy traffic loadings. • Very long-life expectancy (40-60+ years). • Lower maintenance costs over its lifetime. • Good ride quality if paver-laid. • Can be used in areas with poor subgrades.	• High construction cost. • Laying formwork and tying reinforcement is labour-intensive. • Specialist plant required to pave.
8	Continuously Reinforced Concrete Pavement (CRCP). (with/without asphalt surfacing maximum 30mm thick).	High/Very high (10 MCESA to 400 MCESA). Particularly for very high traffic levels (>50 MCESA).	$$$$$	• Greater durability. No problematic joints. • Suitable for very heavy traffic loadings. • Very long life expectancy (40 - 60+ years). • Lower maintenance costs over its lifetime. • Good ride quality. • Can be used in areas with poor subgrades. • In high-temperature range areas, CRCP will perform better than JUCP.	• High construction cost. • Laying reinforcement is labour-intensive. • Specialist plant required to pave. • If it is a concrete surface, noise may be issue.

10.2 Criteria for Overlay Design of Rigid Pavements

10.2.1 Preventive Overlay for Intact Rigid Pavements

The failure of rigid pavements is usually sudden. Ideally, such pavements should be strengthened before failure occurs. Preventive strengthening should be considered when the pavement is nearing the end of its design life or when a traffic increase is expected.

For the design of preventative overlays refer to Section 10.6 of the determination of overlay thickness for the different scenarios. The overlay is designed to mitigate the capacity deficit in carrying future traffic loading.

The pavement condition survey report should be reviewed to confirm that there are no significant defects which may need repairs before the application of the overlay. This includes the existence of cavities and low load transfer efficiency at joints and cracks.

10.2.2 Strengthening Overlay of Deteriorated Rigid Pavements

The following types of deterioration may be identified:

10.2.2.1 Deterioration or Loss of Cohesion

1. **General Loss of Cohesion**

   Degradation of bound layers is, generally, indicated by high deflections and low radii of curvature. Such a pavement can be treated as a flexible pavement and the overlay is designed accordingly.

2. **Surface Disintegration**

   This type of defect affects the surface of cement-treated layers (chemical reaction, the rise of laitance, etc.). It is generally necessary to remove all loose material (and the overlying bituminous surfacing) and then overlay the remaining pavement.

10.2.2.2 Deterioration by Fracture (without loss of cohesion)

1. **Cracking (without faulting)**

   If the rigid pavement is affected only by cracking (no faulting or unstable slabs), the overlay shall be designed so that:

   a. All strains are reduced to the desirable level.

   b. Cracking does not reflect through the overlay.

   Usually, the deciding criterion is the prevention of reflection cracking.

2. **Faulting - unstable and rocking slabs**

   Overlaying is impracticable and pavement reconstruction is required.

10.3 Rigid Pavement Condition Evaluation

Before an appropriate overlay can be designed, it is essential to know the construction and condition of the existing pavement.

Construction information (such as layer types and thicknesses) can be determined from construction records and pavement surveys such as Coring, Dynamic Cone Penetrometer (DCP) testing and Ground Penetrating Radar (GPR).

1
2
3
4
5
6
7
8
9
10
Structural Design of Overlays for Rigid & Semi-rigid Pavement

To determine the condition of the existing pavement appropriate surveys should be carried out – the number and type of surveys will depend upon the road class and observed defects. They should include Visual Condition Survey (VCS), Coring and Dynamic Cone Penetrometer (DCP) testing, deflection test using the Falling Weight Deflectometer (FWD) or Heavy Weight Deflectometer or Benkelman Beam surveys, Ground Penetrating Radar (GPR) survey (this will require measurement of core thicknesses for calibration), drainage surveys, laboratory tests of the cored material, etc.

The aim of the condition surveys is to:

1. Identify and quantify the type and extent of defects and the repairs required. Uniform support makes a concrete overlay perform well, so the pavement should be brought up to a uniform condition. It may be that the type and number of defects identified mean that an overlay would be unsuitable (this is discussed in the next section).
2. Quantify the costs of repairing the identified defects. These can be included in the life cycle analysis to compare different rehabilitation options (discussed later).

10.3.1 Surface Condition Evaluation of Rigid Pavements

The VCS is the main tool in providing the designer with information about the condition of the pavement including whether the pavement needs an overlay and what repairs are required before an overlay.

A visual survey should include the presence of overbridges, power lines, safety barriers, underbridges, etc, as these could affect the decision on whether it is feasible to overlay and raise the road height.

When carrying out a visual survey of an existing CRCP pavement, fine transverse cracks should be ignored. Only significant distresses should be recorded, including longitudinal cracks, medium and wide transverse cracks (3 – 6 mm), faulting (i.e. stepping) (>6 mm), punchouts, spalls (localised breakouts of concrete usually at a joint or a crack) >75 mm, reinforcement corrosion (often combined with spalling), or pumping/erosion of unbound material from beneath the concrete.

Table 10.2 provides the evaluation criteria for visual surface defects.

Table 10.2 Evaluation Criteria for Surface Defects

Surface Defects	Interpretation	Design considerations
Cracks	Shallow cracks	Crack sealing is required
	Deep cracks	Deep treatment is required, which may include milling to the depth reached by the cracks from the top.
	Cracking through the full depth of slabs	FWD Load transfer efficiency (LTE) at joints and cracks: 100 % efficiency is excellent, below 50 % efficiency indicates poor load transfer at joint/crack.
Spalling at joints	Excessive spalling	Joints not functioning and hogging of the slabs is occurring at high temperatures. Joints are dysfunctional and need repair or joint spacing is too wide.
Loss of skid resistance	Excessive loss of surface texture	Shallow surface retexturing or application of non-structural comfort overlays.
Faulting	Failure of the foundation	Replacement of slabs or reconstruction
Pumping	Erosion and water ingress into the foundation	Improve drainage, under-slab grouting treatment or slab replacement.

10.3.2 Structural Condition Evaluation of Jointed Rigid Pavements

Structural condition surveys for jointed concrete pavements usually include Coring and Dynamic Cone Penetrometer (DCP) tests, deflection tests using the Falling Weight Deflectometer (FWD) or Heavy Weight Deflectometer (HWD) or Benkelman Beam.

On multi-lane roads, deflection surveys are usually only carried out in lane 1 in both directions. Surveys in other lanes may be necessary where:

1. Visual defects are very different in other lanes;
2. There is a different construction in other lanes; or
3. The traffic loadings are greater in other lanes.

From the FWD or HWD deflections, the Surface Modulus (SM) for the pavement can be calculated and this can be used to design the overlay (explained below). It should be noted that for a concrete overlay, it is not necessary to examine the residual properties of each layer in the old pavement, but the modulus of the whole existing pavement (that will be left after any weak/cracked asphalt is removed) will be required as this will provide the support for the new concrete overlay.

The structural surveys are carried out in addition to Visual Condition Surveys, which can identify structural issues, such as rocking slabs, cracking, faulting and pumping. Such information will help the designer to decide on appropriate treatments before overlaying.

The structural evaluation criteria are given in Table 10.3 and Table 10.4.

Table 10.3 Structural Condition Evaluation for Jointed Rigid Pavements (JUC/JRC)

Investigation	Pavement characteristics	Design considerations
Cores	Condition and thickness of each of the bound layers.	• Structure and strength of existing pavement • Composition of concrete • Voids in concrete • In-situ densities • Strength of concrete in MPa
	Location, size and condition of steel reinforcement, tie bars and dowel bars.	Design for maintenance rehabilitation and strengthening and overlays
	Depth of cracking and spalling	Milling depth and replacing deteriorated concrete
	Type and condition of sub-base	Coring into the bound base/sub-base can provide information on layer thickness and condition. If required, samples can be tested in the laboratory.
DCP	Profile of the unbound foundation layers	Thicknesses and CBR of unbound layers under the pavement
	Lack of support	• Poor quality materials • Inadequate compaction • Ingress of water • Erosion of underlying layers
FWD Deflections	Poor load transfer at transverse joints.	Deflection Load transfer efficiency (LTE) test at transverse joints and wide cracks. >75% indicates good LTE and values below 50% indicate poor load transfer.
	Voids under the joints	Deflection Void Intercept (VI) test at transverse joints and cracks. VI ≥ 50 microns indicates voiding present.
	Mid-slab deflections. Calculate stiffness and correlate with VCS	See Table 10.4 for expected values of stiffness. Stiffness values will be affected by: cracking, proximity to joints, debonding and poor construction
VCS	Faulting	• Differential settlement and voiding under the pavement. • Rocking of slabs • Failure of the foundation
	Pumping	Erosion of underlying layers and water ingress into the foundation

Table 10.4 Relationship Between the Condition of Pavement and Stiffness (JUC/JRC)

Material Type	Layer stiffness derived from FWD Back-Analysis (GPa)		
	Poor integrity throughout	Some deterioration	Good integrity
PQ concrete	< 20	20-30	>30
Hydraulically bound mixture (HBM)	< 8	8-15	> 15
Unbound foundation	< 0.1		≥ 0.1

10.3.3 Structural Condition Evaluation of Continuously Reinforced Rigid Pavements

This section covers the structural condition evaluation for continuously reinforced rigid pavements (CRCB/CRCP). The evaluation criteria are given in Table 10.5.

Table 10.5 Structural Condition Evaluation of Continuously Reinforced Rigid Pavements

Investigation	Pavement Characteristics	Design Considerations
Cores	Condition and thickness of each of the bound layers	• Structure and strength of existing pavement • Composition of concrete • Voids in concrete • In-situ densities • Strength of concrete in MPa
	Presence, size and location of steel reinforcement	Design for rehabilitation, strengthening and overlay
	Depth of cracking and spalling	Milling depth and replacing deteriorated concrete
	Type and condition of bound foundation	Coring into the bound base can provide information on the strength of the base, voids under the concrete and access for DCP tests and sampling of materials
	Assessment of mechanical interlock across cracks	Load transfer efficiency (LTE) to be assessed: 100% efficiency if the difference in deflections on either side = 0. Below 50% efficiency indicates failure.
	Condition of materials	Composition and deterioration of pavement materials
DCP	Profile of the unbound foundation layers	Thicknesses and CBR of unbound layers under the pavement
	Lack of support	• Poor quality materials • Voids below the pavement • Inadequate compaction • Ingress of water
Deflection profiles	Condition of pavement layers	Reduced stiffness indicating poor condition of pavement layers
	Deflections – stiffness *(Correlate with VCS)*	See Table 10.4 for expected values of stiffness. Stiffness is affected by: • Cracking • Proximity to joints • Debonding • Poor construction
VCS	Cracking	• Transverse cracks at spacing less than 1m • Transverse cracks with widths greater than 1mm • Longitudinal cracks • Areas of polygonal cracking • Loose or missing blocks of concrete (punch outs) • Crack bifurcations • Failing repairs • Spalling
	Faulting	Differential settlement and voiding under the pavement. Failure of the foundation
	Pumping	Erosion and water ingress into the foundation

Use values given in Table 10.4 for the relationship between the condition of the pavement and stiffness for CRCP. Similar stiffnesses derived from FWD are applicable.

10.3.4 Failure Criteria of JUC

For JUC pavements, the definition of failure for an individual bay (i.e. slab = a length of concrete separated by a transverse contraction or expansion joint) is:

1. A crack of width ≥ 0.5 mm, crossing the bay longitudinally or transversely.
2. Longitudinal and transverse cracks intersecting; starting from an edge and > 0.5 mm wide, and each longer than 200 mm.
3. A corner cracking wider than 1.3 mm and more than 200 mm radius.
4. A bay with pumping at a joint or edge.
5. A bay with significant settlement.
6. A replaced or structurally repaired bay.

It is suggested that a road can be considered to have reached ‘failure’ when 50% of slabs have failed using the definitions above.

However, it should be noted that even when a road has reached this ‘failed’ state, it may still be useable and safe to drive on but is likely to need an upgrade in the relatively near future.

10.3.5 Failure Criteria of CRCP

For CRCB/CRCP pavements, defects that are considered ‘significant’ are given below:

1. Transverse cracks at spacings less than 1m.
2. Transverse cracks with widths greater than 1mm.
3. Longitudinal cracks.
4. Areas of polygon cracking.
5. Loose or missing blocks of concrete (punchouts).
6. Crack bifurcations.
7. Failing repairs.
8. Spalling.

For a CRCB/CRCP pavement, a severe level of deterioration (i.e. failure is defined as greater than 15 significant defects per 100m lane length.

10.4 Characterisation of Rigid and Semi-Rigid Pavements

10.4.1 Determination of Uniform Sections

Rigid and semi-rigid pavements in good condition exhibit low deflections and high radii of curvature.

Homogeneous sections can be represented by the characteristic radius of curvature R_{10} given by Equation 10.1:

$$R_{10} = R - 1.3S$$

Equation 10.1

Where R_{10} = characteristic radius of curvature, R = the mean and S = standard deviation.

Alternatively, uniform sections can be determined using the surface modulus (SM) of the pavement instead of the radius of curvature.

1
2
3
4
5
6
7
8
9
10
Structural Design of Overlays for Rigid & Semi-rigid Pavement

10.4.2 Determination of the Surface Modulus of Existing Pavement

In many design methods, the Equivalent Surface Foundation Modulus (*ESFM*), also known as the Surface Modulus (*SM*) is required to design the concrete overlay.

The ESFM (*SM*) is defined as the modulus of a uniform elastic foundation that would give the same deflection under the same wheel load as the actual road structure.

The Surface Modulus (*SM*) can be calculated from FWD or HWD or Benkelman Beam deflection data at each test point, using the following equations devised by Ullidtz & Peattie (1980):

The Surface Modulus (E_0) at the top of the pavement is given by Equation 10.2:

$$E_0 = 2(1 - \mu^2)\,\sigma_0 \frac{a}{\delta_0}$$

Equation 10.2

Where,

E_o = The surface modulus at the centre of the loading plate (MPa)

μ = Poisson's ratio (usually = 0.35 for an asphalt material or = 0.45 for an unbound material)

σ_o = The contact pressure under the loading plate (kPa)

a = The radius of the loading plate (mm)(usually 300mm)

δ_0 = The central deflection, at a distance 0 (in microns).

It should be noted that the Surface Modulus calculated will be at the 'as-measured' temperature. The stiffness of asphalt layers, unless severely cracked, should be adjusted to the standard reference temperature of 20ºC using the following equation:

Equation 10.3 is used to adjust asphalt layer stiffness to standard temperature (20ºC)

$$E_{20} = E_T \times 10^{(0.0003 \times (20 - T)^2 - 0.002 \times (20 - T))}$$

Equation 10.3

Where,

E_{20} = Stiffness at 20ºC.

E_T = Stiffness at temperature T.

T = Temperature of the asphalt (at a depth of 100 mm) at the time of testing.

The surface modulus can be obtained at the depth corresponding to the surface foundation using the equation below.

Equation 10.4 is used for calculating the Surface Modulus at equivalent depth $E_0(r)$:

$$E_0(r) = (1 - \mu^2)\,\sigma_0 \frac{a^2}{r.\delta_r}$$

Equation 10.4

Where,

E_0 = Surface modulus at the centre of the loading plate (MPa).

$E_0(r)$ = Surface modulus at distance r from the central load (MPa).

μ = Poisson's ratio.

σ_0 = The contact pressure under the plate (kPa).

a = Radius of the loading plate (mm).

δ_r = The deflection at a distance r from the central load (microns).

The next stage would be the determination of the foundation class.

10.4.3 Determination of Foundation Class

The design of pavement foundations is given in Table 10.6 (see RDM 3.3 and 3.5).

Table 10.6 Foundation Classes

Native Subgrade Class	Improved Subgrade/Capping		New Subgrade Class	Foundation Class
	Material	Minimum Thickness (mm)		
S1 (2-5 % CBR) Median = 3.5 % CBR	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 (5-10 % CBR) Median = 7.5 % CBR	G10	150	S3	F1
	G14	150	S3	F1
	G14	175	S4	F2
S3 (7-13% CBR) Median = 10 % CBR	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 (10-18% CBR) Median = 14 % CBR	G23	150	S5	F3
	G45	200	S6	F4
	HIG100	200	S6	F4
	BSM50	225	S6	F4
S5 (15-30% CBR) Median = 22.5 % CBR	G45	150	S6	F4
	GCS/BSM50	275	S6	F4
	HIG160/HMS1	125	S6	F4 (bound)
	HIG160	250	N/A	F5 (bound)
	HMS1	225	N/A	F5 (bound)
S6 (30-60 % CBR) Median = 45 % CBR	GCS	250	N/A	F5
	HIG160	200	N/A	F5 (bound)
	HMS1	175	N/A	F5 (bound)
	BSM100	150	N/A	F5 (bound)

Adapted Table from RDM 3.3.

Notes:

1. *BSM = Bitumen Stabilised Material, G = Natural Gravel, GCS = Graded Crushed Stone, HIG = Hydraulically Improved Granular Material, HMS = Hydraulically Modified Stone.*

2. *For concrete pavements (JUC, JRC, RCC, CRCB, CRCP), only foundations F4 (bound) and F5 (bound) may be used and the top-most capping layer must be HIG160, HMS1, BSM100, or higher quality BOUND material. For low-volume roads, foundation class F1 may be used without a bound capping. These are highlighted in Table 10.6 & Table 10.7.*

3. *S1 and S2 subgrades must first be improved to minimum of S3 (F1) or S5 (F3) subgrade class before final improvement to F4 (bound) and F5 (bound).*

1
2
3
4
5
6
7
8
9
10
Structural Design of Overlays for Rigid & Semi-rigid Pavement

Table 10.7 Foundation Classes for Rigid Pavements

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

10.5 Estimation of Residual Life of Rigid Pavements

For most concrete pavements, the residual life can be calculated using the fatigue life of concrete. Also, see Section 10.10.

For Roller Compacted Concrete (RCC), the design life can be calculated using Equation 10.5.

Use fatigue Equation 10.5 to calculate the design life of RCC:

$$\textit{Design life, } T = \frac{e^{\frac{\textit{Stress Ratio} - 0.9157}{-0.039}}}{10^6}$$

Equation 10.5

Stress Ratio = $\frac{\epsilon_t}{f}$

Where,

T = Design life (MCESA).

e = Base of the natural logarithm.

ε_t = Tensile stress at the bottom of the concrete pavement.

f = Flexural strength in, MPa.

Stress ratio = tensile stress at the bottom of the RCC due to a standard wheel load divided by the flexural strength of the RCC.

10.6 Concrete Overlay Design - UTRCP for Low-Volume Roads

Ultra-thin reinforced concrete pavement (UTRCP) is primarily a rehabilitation treatment for low-volume urban or rural roads (0.1 MCESA to 3.0 MCESA) with poor surfacings, where the pavement structural capacity does not need to be increased. They have been applied to existing asphalt and unpaved roads.

UTRCP involves removing deteriorated upper asphalt layer(s) and applying a thin (50 mm thick) lightly reinforced concrete overlay. This pavement technology is becoming increasingly popular in solving low-volume road problems in rural and urban road rehabilitation and surfacing.

It should be noted that UTRCP is not the same as UTCRCP (Ultra-Thin Continuously Reinforced Concrete Pavement) which uses higher strength concrete (80 MPa-100 MPa), thicker and more frequent steel reinforcement and steel fibres

For low-volume roads, UTRCP involves removing most, if not all, of the deteriorated asphalt and adding a 50 mm thick layer of 30 MPa concrete, with light steel wire reinforcement placed at mid-layer depth. Originally the reinforcement comprised 5-6mm diam. steel wire welded in a 200 mm x 200 mm mesh, but after analysing the performance of previous UTRCP schemes, Brink (2016) proposed changing this to 4mm thick steel wire in a 100 mm x 100 mm mesh to prevent block cracking. The concrete is laid continuously in strips between formwork (approximately 3m wide) and compacted using a vibrating beam/screed. The surface is then textured with a transverse brushed finish. Reinforced concrete ground anchors (400 mm deep) are constructed at both ends of straight sections (which may be up to several hundred metres long) to restrict lateral movement at the ends.

UTRCP can be built with stronger concrete (120 MPa-140MPa) and 4-8 mm steel in a 50 mm x 50 mm mesh (Kilian, 2009).

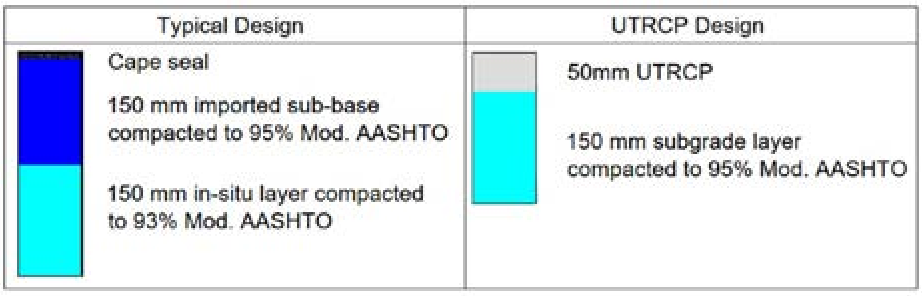
Materials used for the UTRCP consist of 13 mm aggregate; graded river sand; CEM I – 42.5 N; Reference 200 mm steel mesh; Plasticiser and Polypropylene fibres. 30 MPa concrete with a 150 mm slump. The given (cement to sand and stone aggregate = 1:1.5:3 by volume).

As the overlay is CRCP, there are very few transverse joints (in contrast to a conventional JUC pavement) and so most of the problems associated with joints have been removed.

The continuous concrete layer functions as a stretched sheet of material, as it is tied to ground anchors that prevent movement at both ends. A substantial amount of the traffic loading is taken up as tensile forces (by the steel reinforcement) so less of the wheel load is directly transferred to the underlying layers. As a result, less concrete thickness is needed to achieve the same long-term resistance to traffic as with conventional road construction methods.

For example, a pavement comprising: (1) A cape seal on (2) 150 mm thick imported sub-base (compacted to 95% Mod AASHTO)) can be replaced with a 50 mm UTRCP sitting on (3) a 150 mm thick in-situ subgrade compacted to 93% Mod AASHTO. This is illustrated in Figure 10.1.

Figure 10.1 Comparison of Conventional Pavement with UTRCP



There are several key points to note regarding UTRCP:

1. The advantage of such a thin concrete layer for the construction of new slip lanes and lane widening in urban areas is that the existing utility services (which usually get in the way of road widening), can be left intact and the widening can be built with minimum disruption to these services.
2. The load spreading ability of the continuously reinforced concrete can reduce stresses on the lower layers, which means that a localised weak area in the underlying layer is less likely to develop into a pothole than a conventional asphalt pavement.
3. Where an unpaved road has a UTRCP overlay, an additional lime-stabilised sub-base layer beneath the CRCP is constructed for roads carrying additional HGVs from a nearby quarry.
4. Production rates are reported to be comparable with paver-laid asphalt when using ready-mix concrete and approximately 15 workers.
5. **Costs** - The construction costs of 45 mm AC and 50 mm UTRCP are comparable and block paving is generally more expensive than UTRCP.
6. For remote areas where asphalt or block paving is not available, the 50 mm UTRCP option provides further cost savings. In addition, the omission of a sub-base layer (and hence cost reduction) makes this an even more competitive option to conventional surfacing.
7. A further possible cost advantage in the longer term is that a UTRCP does not usually need continuous rehabilitation, whereas conventional roads will require maintenance every 7 to 10 years, owing to the 'age hardening' of the bituminous surfacing, etc.

1
2
3
4
5
6
7
8
9
10
Structural Design of Overlays for Rigid & Semi-rigid Pavement

10.7 Empirical Concrete Overlay Design

The Empirical Concrete Design Method is derived from the Standard UK Method for concrete overlay design, which effectively treats the existing pavement as a sub-base and provides a new concrete slab on top of this, usually with an asphalt interlayer to stop cracks from reflecting into the new overlay. There is no reduction in concrete thickness to account for the inherent strength of the existing pavement. The minimum overlay thickness allowable is 200 mm.

10.7.1 Design Thickness of JUC (URC) Overlay

There are only a few occasions when a JUC or JRC overlay would be used to overlay an existing pavement. The main reason is that most of the issues with jointed concrete pavements occur at the joints, so a rehabilitation scheme would aim to minimise or remove transverse joints, for example by using a CRCB/CRCP overlay.

A JUC or JRC overlay would be used to overlay an existing pavement when:

1. Overlaying a failed asphalt road with low to medium traffic levels, where the cost of a CRCP overlay would be too high.
2. Overlaying an existing JUC/JRC pavement. A bonded concrete overlay (i.e. no interlayer) would need to be carried out, with every joint in the existing road replicated in the new overlay at the same locations. Repairs to the existing pavement. would need to be carried out before the overlay. An unbonded CRCP overlay with interlayer would probably be a better option.

To design a JUC overlay, the new pavement design method can be used (see RDM 3.5: Rigid Pavement Design). The following equations are used:

Design Thickness of JUC (URC) pavement (no tied lane/shoulder or 1m edge strip), Equation 10.6:

$$Ln(H_1) = \frac{Ln(T) - 3.466Ln(R_c) - 0.484Ln(E) + 40.483}{5.094}$$

Equation 10.6

Effect on design thickness of URC pavement with a tied lane/shoulder/edge strip), Equation 10.7:

$$H_2 = 0.934H_1 - 12.5$$

Equation 10.7

Where,

H_1 = Thickness (mm) of the concrete slab without a tied lane or 1m edge strip.

H_2 = Thickness (mm) of the concrete slab with a tied lane or 1m edge strip.

In or Ln = Natural logarithm.

T = Design life (MCESA).

R_c = Mean compressive cube strength (N/mm^2 or MPa) at 28 days.

E = Foundation stiffness (MPa) related to the foundation class, where,

E = 200 MPa for foundation class F4 (bound).

E = 400 MPa for foundation class F5 (bound).

Notes:

1: Minimum slab thickness (H_1) is 150mm.

2: Maximum design traffic (T) is 400 MCESA.

3: Load-induced stresses at slab corners are greater than in the slab centre, necessitating dowel bars to distribute loads between slabs.

10.7.2 Design Thickness of JRC Overlay

A JRC overlay would not normally be constructed – see text in Section 10.7.1. This would be a bonded overlay over an existing JRC (no interlayer with joints in the new overlay directly above those in the existing pavement) or an unbonded overlay with the interlayer over any existing pavement.

To design a JRC overlay, the new pavement design method can be used – see RDM Volume 3, Part 5: Rigid Pavement Design The following equations are used:

Design Thickness of JRC pavements (no tied lane or 1m edge strip), Equation 10.8:

$$Ln(H_1) = \frac{Ln(T) - R\text{-}\ 3.171Ln(R_c) - 0.326Ln(E) + 45.150}{4.786}$$

Equation 10.8

Effect on design thickness of JRC pavement with a tied lane or 1m edge strip, Equation 10.9:

$$H_2 = 0.934H_1 - 12.5$$

Equation 10.9

Where,

H_1 = Thickness (mm) of the concrete slab without a tied lane or 1 m edge strip.

H_2 = Thickness (mm) of the concrete slab with a tied lane or 1 m edge strip.

In or Ln = Natural logarithm.

T = Design traffic (MCESA)

R_c = Mean compressive cube strength at 28 days (N/mm^2 or MPa)

E = Foundation stiffness (MPa) related to the foundation class:

E = 200 MPa for foundation class F4 or E = 400 MPa for foundation class F5.

R = 1.418$Ln(Rx)$.

Where, Rx = cross-section area of longitudinal steel reinforcement per metre width of slab (mm^2/m):

R = 8.812 for Rx = 500 mm^2/m reinforcement.

R = 9.071 for Rx = 600 mm^2/m reinforcement.

R = 9.289 for Rx = 700 mm^2/m reinforcement.

R = 9.479 for Rx = 800 mm^2/m reinforcement.

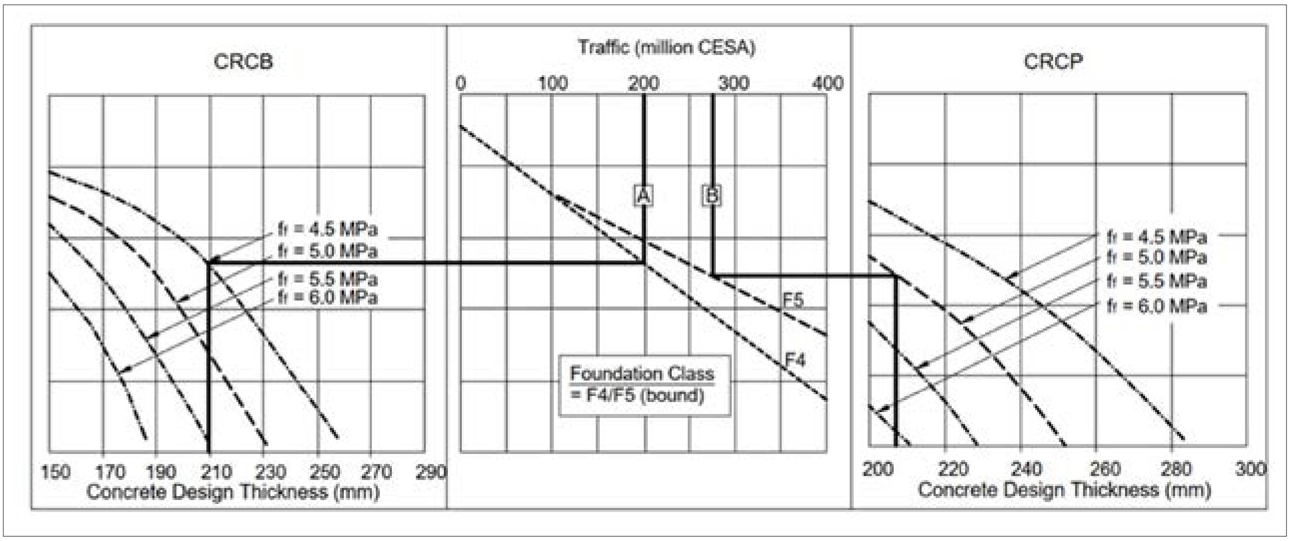
10.7.3 Design Thickness of CRCB/CRCP Overlay

A CRCB/CRCP overlay would be the main type of concrete overlay and could be used over any existing pavement type. It would normally be an unbonded overlay with an asphalt interlayer.

To design a CRCB/CRCP overlay, the same method should be used when designing a new pavement (see RDM 3.5), i.e. the following chart is used to determine the concrete thickness above the foundation, Figure 10.2.

1 2 3 4 5 6 7 8 9 10 Structural Design of Overlays for Rigid & Semi-rigid Pavement

Figure 10.2 Determination of CRCP and CRCB Thicknesses Over the Pavement Foundation



Notes:
For foundation class F4 (bound), E = minimum 200 MPa,
for foundation class F5 (bound), E = minimum 400 MPa.
Thicknesses are to be rounded up to the nearest 5 mm.
The foundation under a concrete pavement must be non-erodible (i.e. bound).

10.8 Modified Empirical Concrete Overlay Design

The most common type is a CRCP overlay. The Modified Empirical Concrete Overlay Design Method is also derived from the Standard UK Method for concrete overlay design, which effectively treats the existing pavement as a sub-base and provides a new concrete slab on top of this but allows a thinner minimum thickness. There is no reduction in concrete thickness to account for the inherent strength of the existing pavement.

The Modified Empirical Concrete Overlay Design Method allows CRCP overlay thicknesses to be a minimum of 150 mm.

For cases where most or all of the asphalt or concrete on the existing road is removed and the surface modulus of the layer to be overlaid is below 500 MPa, the minimum thickness is kept at 200 mm so that it does not go below the thickness required from the new pavement construction design nomograph given in Figure 10.2.

This can be used for existing asphalt or concrete pavements.

10.8.1 Design Thickness of CRCP Overlay

A Continuously Reinforced Concrete Pavement (CRCP) overlay can be considered for all flexible, rigid or composite pavements that would otherwise require reconstruction. Cost savings can occur as the existing pavement and foundation are retained to form part of the foundation of the new road structure.

To calculate the CRCP concrete overlay thickness, use equations Equation 10.10 and Equation 10.11 below. These equations (from TRL630) form the basis of the design graphs for new CRCP (from DMRB CD226); however, the equations allow the ESFM of the existing pavement to be taken into consideration to reduce the thickness of the CRCP overlay.

$$Ln(H_1) = \frac{Ln(\mathrm{T}) - 3.17Ln(f_f)^{1.55} - 0.33Ln(SM) + 30.47}{4.79}$$ **Equation 10.10**

Where,
H_1 = CRC Slab Thickness **WITHOUT** a tied shoulder (mm).
T = Cumulative traffic loading (million CESA).
f_f = Mean flexural strength @28 days.
SM = Equivalent Surface Foundation Modulus (MPa) at 20ºC.

For CRCP **WITH** a tied shoulder/edge strip, the CRCP concrete thickness (H_2) can be calculated using Equation 10.11 below:

$$H_2 = 0.934\ H_1 - 12.5$$

Equation 10.11

Where,
H_1 = CRC Slab Thickness **WITHOUT** a tied shoulder (mm).

It should be noted that if the existing pavement to be overlaid contains asphalt layers then (unless the layer is severely cracked), the stiffness should be adjusted to the standard reference temperature of 20ºC using Equation 10.3.

If the existing pavement is to be overlaid as it is (i.e., without any removal of the deteriorated asphalt), the stiffness modulus can be calculated at each FWD/LWD test location.

The pavement can be split into homogenous lengths based on the surface modulus (SM) of the pavement, central deflection (d_0), or the radius of curvature using the CUSUM method or a similar technique (see Section 10.4.1).

For each homogenous length, the 15th percentile EFSM can be calculated (to provide a safety factor) and this value is used in Equation 10.4.

The overlay thickness can then be calculated for each homogenous length (using the above equations). The overlay can then be constructed using (i) a separate overlay thickness for each homogenous length or (ii) use the thickest calculated overlay over the whole site.

10.9 Structural Deficit Method of Concrete Overlay Design

This method involves the determination of the thickness of the required concrete pavement over the existing foundation for future traffic loading assuming there is no existing concrete pavement (refer to RDM 3.5 for details and design considerations) and subtracting the thickness of the existing concrete pavement. This is based on both the UK and AASHTO design principles.

1. Using the surface modulus of the foundation and foundation class and the design traffic obtained in Section 10.6, determine the concrete pavement thickness required for new construction.
2. Generally, the concrete pavement thickness required for future traffic over the foundation would be thicker than the existing concrete pavement. If milling of the existing concrete pavement is required to remove defects, the remaining thickness is considered for the design.
3. Subtract the concrete thickness required over the foundation for new construction calculated in (1) above from the thickness of the existing pavement determined in (2) above to get the structural deficit. If the value is positive then no overlay is required. A negative value would be the overlay thickness over the existing concrete pavement.
4. The minimum thickness requirements for construction purposes for the different rigid pavement options should be adhered to. See Table 10.8 for guidance on the expected thicknesses for non-structural and structural concrete overlays.
5. If the existing concrete pavement has fully deteriorated or disintegrated, it shall be considered to be part of the foundation.
6. If the existing pavement is adequate for future traffic then a bonded thin overlay of 50 mm to 75 mm may be applied to improve on the riding quality when necessary.

Table 10.8 Minimum Thicknesses that are Required for the Constructability of Concrete Pavements

Purpose	Thicknesses
Non-structural overlays	50 – 100 mm
Structural overlays	100 – 250 mm

10.10 Mechanistic-Empirical Design Method of Concrete Overlays

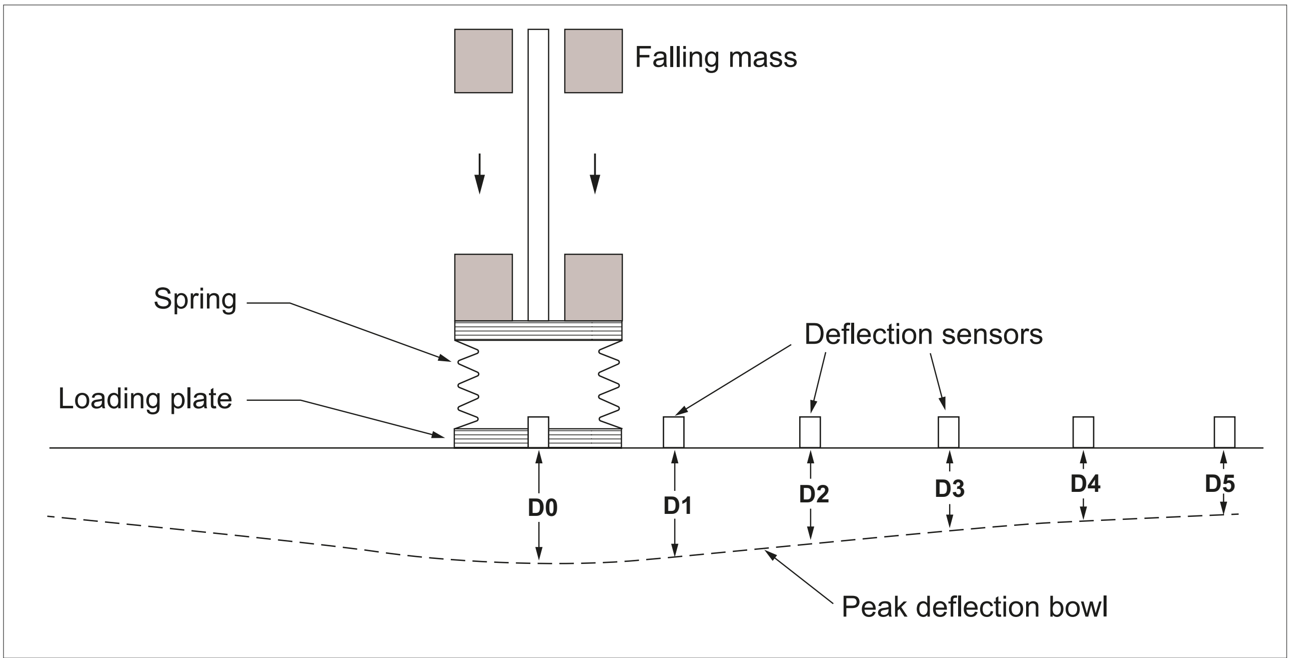
This section covers the overlay design of rigid pavements using the Mechanistic-Empirical Design Method based on fatigue principles. The method is based on the India/AASHTO mechanistic design approach.

This primarily considers the properties, stiffness and thickness of each layer of the pavement as well as that of the proposed overlay, using multi-layer linear elastic modelling. The properties of the concrete include an assumed layer stiffness of the intact or fractured slabs.

The process is similar to the Mechanistic Design of flexible pavements and the following stages shall be followed:

1. The following deflections should be considered, Figure 10.3:

Figure 10.3 Configuration of Deflections for Test on Rigid Pavements



The radial distances to be considered in rigid pavement evaluation are given in Table 10.9.

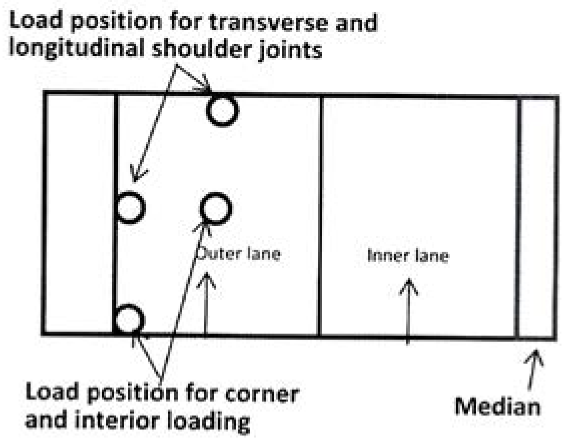
Table 10.9 Deflections and Radial Distances Required in the Design

Deflection	Radial Distance
D0	0
D1	300
D2	600
D3	900

2. Generally, the plate diameter should be 300 mm-450 mm and a 300 mm diameter is preferred.
3. The falling weight is generally 50 kg-350kg but for concrete pavements 200 kg-700kg weight is preferred.

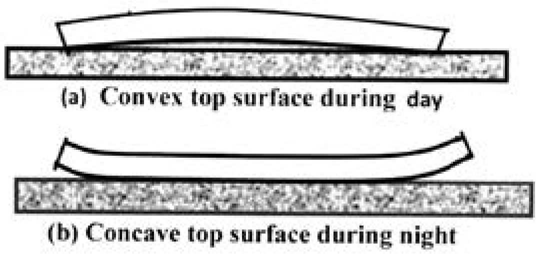
4. The range of height of fall of the mass 100 mm-600 mm
5. The load should be generally 40 kN and 60 kN or preferably higher for rigid pavements.
6. Some of the tests should be carried out with the load at Corners, interior, transverse and longitudinal shoulder joints, Figure 10.4.

Figure 10.4 Test Positions for Rigid Pavements



7. The deflections (FWD) should be collated and cleansed of all errors.
8. Tests should not be carried out when the temperature is > 40°C. This is because of temperature-induced warping of the concrete, Figure 10.5.

Figure 10.5 Shapes of Rigid Pavements During Day and Night



9. Back calculation shall be carried out to determine the E-moduli of the different layers. A guide on the expected E-moduli. Indicative values are given in Table 10.4.
   a. Determine the area parameter, (A) – the area parameter of the deflection bowl is calculated using the following Equation 10.12:

$$A = 6\left[1 + 2\left(\frac{D1}{D0}\right) + 2\left(\frac{D2}{D0}\right) + \left(\frac{D3}{D0}\right)\right]$$

Equation 10.12

Where,

A = Area parameter of the deflection basin.

D = Deflections at designated radial distances, Table 10.9.

The value of A is approx. 11.8 for a single-layer elastic half-space (pavement and soils have the same elastic modulus).

For an extremely rigid layer with high elastic modulus where $D0=D1=D2=D3$, A=36.

1 2 3 4 5 6 7 8 9 10 Structural Design of Overlays for Rigid & Semi-rigid Pavement

b. Determine the radius of relative stiffness, (l) using Equation 10.13:

$$l = \sqrt[4]{\frac{Eh^3}{12(1-\mu^2)k}}$$

Equation 10.13

Where,

l = Radius of relative stiffness, cm.
E = Modulus of elasticity of concrete, MPa.
H = thickness of concrete slab, cm.
M = Poisson's ratio.
k = Modulus of subgrade reaction.

c. Normalise deflections using the temperature region of the area (climatic condition), see Chapter 9.

d. Determine the subgrade modulus using the following equation:

$$k_i = \frac{Pd_i}{l^2 D_i}$$

Equation 10.14

Where,

i = 1,2,3,4.
l = Radius of relative stiffness, mm.
P = Load, kN.
D_i = Measured deflections at various radial distances, mm.
D_i = Normalised deflections at various radial distances, mm.

K value for pavement design should be 50 % of that determined by FWD since only the static elastic modulus of the subgrade reaction is to be used for the pavement design.

e. Determine the elastic modulus of concrete, (E_c) using Equation 10.15:

$$E_c = \frac{12(1-\mu_c)kl^4}{1000h^3}$$

Equation 10.15

Where,

μc = Poisson's ratio of concrete.
h = Thickness of the concrete layer in mm.
l = Radius of relative stiffness, mm.
k = Modulus of subgrade reaction, MPa.

The strength of concrete can be obtained from the following equation:

$$f_c = \left(\frac{E_c}{5000}\right)^{0.5}$$

Equation 10.16

Flexural strength (f_{mr}) can be obtained from the following equation:

$$f_{mr} = 0.7(f_c)^{0.5}$$

Equation 10.17

The values of E_c, f_c and f_{mr} are concrete properties at the age at which the FWD tests are performed. Exact values can be obtained from tests on concrete cores in the laboratory.

10. Determine the fatigue life of concrete - fatigue relationships given in Equation 10.18, Equation 10.19 and Equation 10.20 below shall be used to determine the fatigue life of the pavement:

$$N = \left[\frac{4.2577}{SR - 0.4325} \right]^{3.268} \quad \text{for } 0.45 \le SR \le 0.55$$

Equation 10.18

$$Log\ N = \frac{0.9718 - SR}{0.0828} \quad \text{for } SR > 0.55$$

Equation 10.19

Where,

N = fatigue life (total number of CESA)

SR = ratio of load stress and modulus of rupture of concrete.

$$SR = \frac{P}{f_{mr}}$$

Equation 10.20

11. Determine the cumulative fatigue damage CFD during the design period using Equation 10.21:

$$CFD = \sum_{i=1}^{k} \frac{n_i}{N_i} (10 \text{ am to } 4 \text{ pm}) + \sum_{i=1}^{k} \frac{n_i}{N_i} (0 \text{ am to } 6 \text{ am}) + \sum_{i=1}^{k} \frac{n_i}{N_i} (\text{remaining hrs})$$

Equation 10.21

Contribution to bottom-up cracking is computed for the period 10 am to 4 pm.

Contribution to top-down cracking is computed for the period 0 am to 6 am.

The sum of fatigue life for both should be less than 1.0. to make the design more conservative.

12. The fatigue life can be determined using software.

13. If the residual life is greater than the design life, then strengthening is not required and pavement maintenance should be carried out, see Chapter 15.

14. If the residual life is less than the design life, strengthening is required.

15. Overlay thickness – using the E-moduli obtained from the back calculation, the layer thicknesses and Poisson's ratios and the fatigue equations in the software as well as a selected overlay option and initial estimated thickness carry out the iterative process to determine the overlay thickness using software.

16. The overlay thickness – can also be calculated manually by substituting SR as a function of the required concrete thickness in the fatigue equation and determining the design thickness (h_d) by solving for h in the equation.

10.11 Considerations for Composite and Other Overlays Options

Different overlay options can be designed using the same design process given in Section 10.5.7. The composite overlay options are given in Table 10.10. Types of composite overlays are shown in Figure 10.6 to Figure 10.9.

Table 10.10 Composite Overlays

Item No.	Overlay Options	Design Considerations
1	Thin concrete overlay	• Should be bonded to the existing pavement. • No joints in wheel path. • Place joints at the same positions as the existing pavement • Existing pavement should be blown or sandblasted or milled to ensure a good bond. • Should be applied on fairly intact pavements. • All repairs should be carried out before application of the overlay. • General thickness requirement should be 50 mm minimum.
2	Thick structural overlay	• Generally, unbonded but slight bonding may be advantageous. • Joints to be cut through the overlay and approximately 12 mm into the existing pavement. • To be applied when the temperature of the existing pavement is no more than 50°C.
3	Concrete overlay on asphalt pavement	• Mill asphalt surface to remove irregularities and deteriorated materials. • Patching carried out using concrete should not be connected to the overlay to allow uniform thermal movement of the pavement layers. • The overlay should not be applied when the temperature of the existing pavement is above 50°C.
4	Asphalt overlay on concrete pavement	• Include a tack coat for bonding asphalt to the concrete. • For jointed concrete, include joints in the asphalt at the same positions as the concrete and treat with the same sealant. • Include a regulating layer to prevent reflective cracking. • Use aggregate with similar thermal expansion characteristics.

Figure 10.6 Overlay Over Granular Subbase

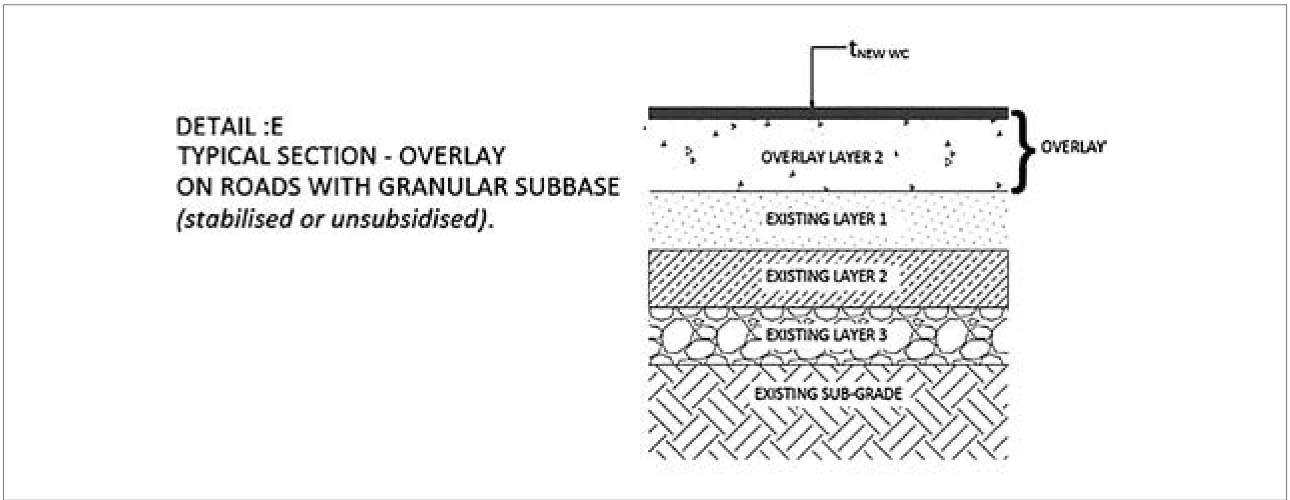


Figure 10.7 Concrete Overlay on Existing Concrete Pavement

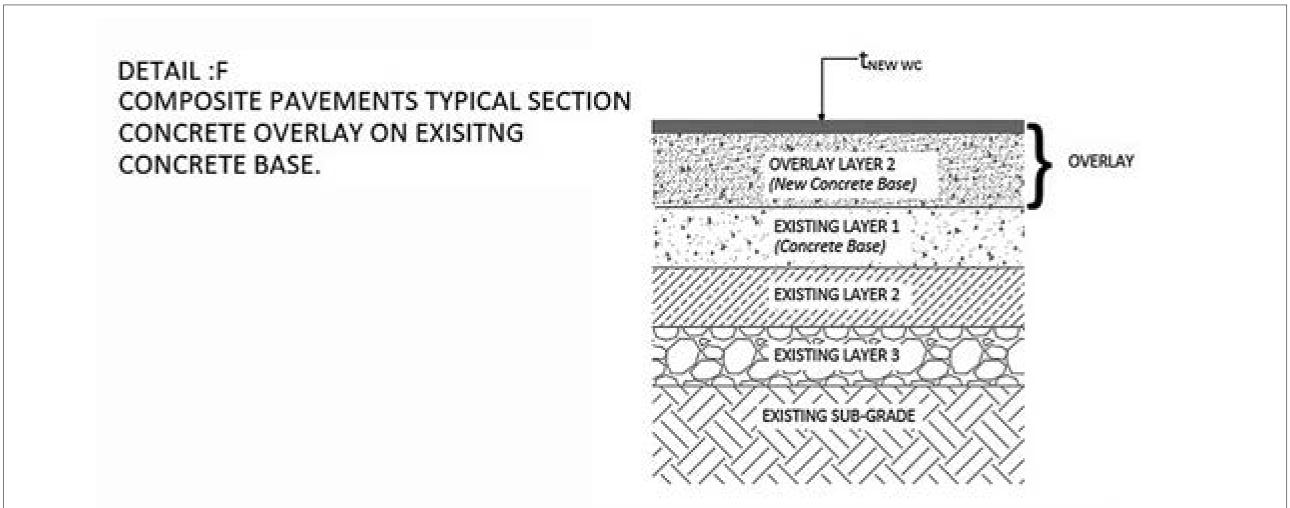


Figure 10.8 Asphalt Overlay on Existing Concrete Pavement

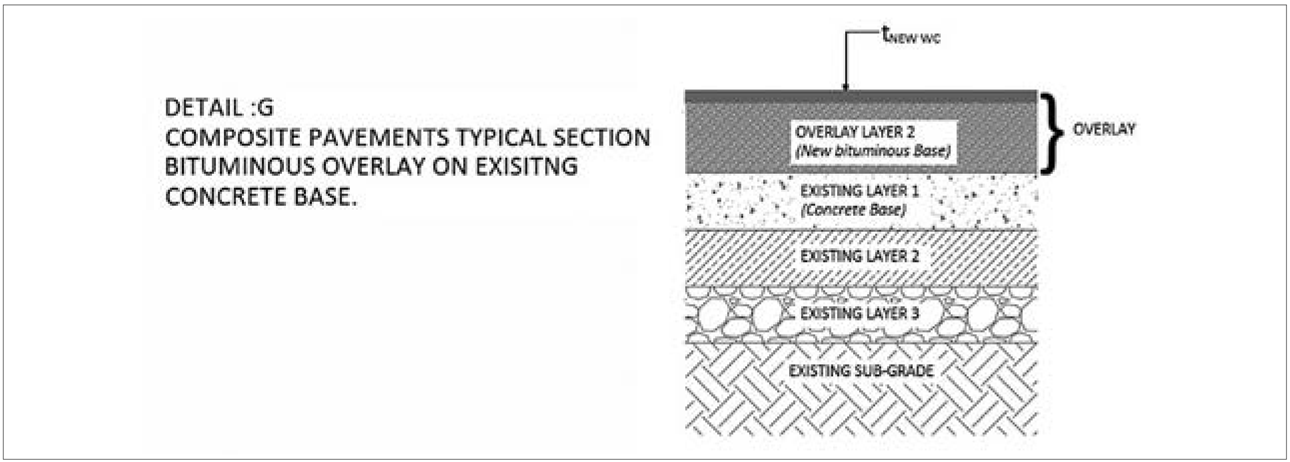
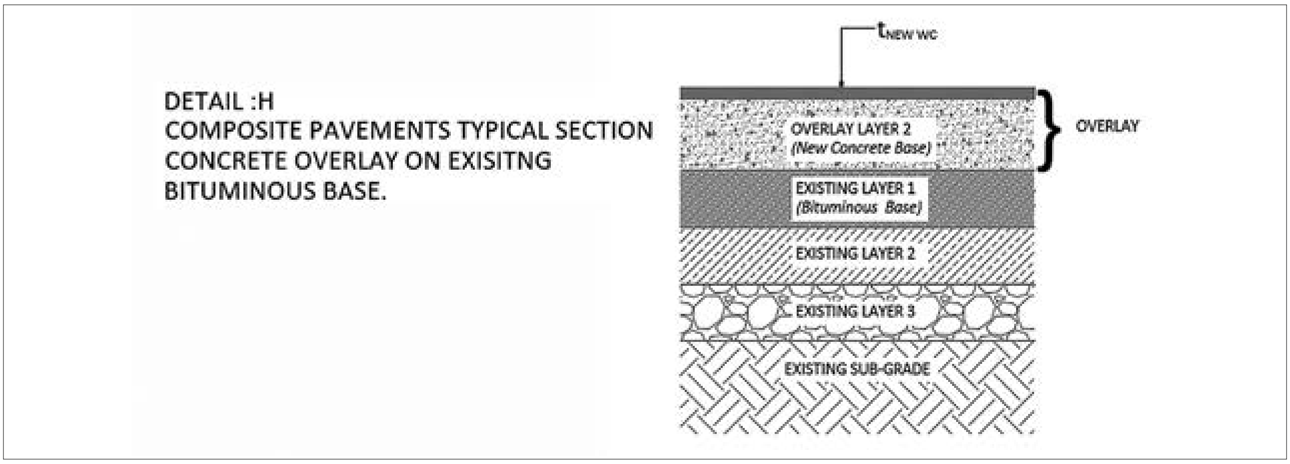


Figure 10.9 Concrete Overlay on Existing Asphalt Pavement



11 Other Rehabilitation and Construction Techniques

11.1 General

Besides overlays and drainage improvements, several other corrective measures, although not directly related to structural adequacy, contribute to restoring a pavement to a satisfactory condition. This chapter provides general guidance on these techniques and deals with some aspects of pavement reconstruction.

11.2 Surface Treatments

Surface treatments are used for the following reasons:

1. To renew and add life to the old bituminous surfacing.
2. To stop the disintegration (ravelling and stripping) of defective surfacing.
3. To seal small cracks and waterproof the surface

Skid resistance, the most satisfactory method of correcting a slippery surface is to apply a surface dressing using hard, angular and non-polishing aggregates.

Surface treatments are economical and may be applied to any paved road.

11.2.1 Surface Dressing

Surface dressing is covered in Volume 3 'Pavement Design for New Roads'.

Classical binders (cut-backs and emulsions) are suitable only for traffic up to 6,000 vehicles per day on 2 lanes. For heavier traffic, special binders, containing polyvinyl chloride or rubber, are required.

11.2.2 Emulsion Slurry Seal

Emulsion slurry seals are covered in RDM Vol 3.

1. Type I (fine) is employed mainly to seal cracks.
2. Type II (normal) is normally used for resealing an existing surfacing.
3. Type III (coarse) is reserved for heavy traffic and for areas where good skid resistance is required.

11.2.3 Fog Spray or Surface Enrichment

Fog spray/seal is a very light application of diluted bitumen emulsion aimed at rejuvenating old surfacings and sealing small cracks and surface voids.

The bitumen emulsion shall be slow-breaking anionic A3 or cationic K3. To rejuvenate existing old surfacing, the residual bitumen should be soft and penetrating.

Where fog sprays are used just as a sealant on the existing surface, harder residual bitumen (50/60 polymer-modified bitumen) should be used (see RDM Vol 3 for specifications). Polymer-modified emulsion can be applied without dilution with water. Where dilution is required at a rate of 1:1 or 2:3 parts emulsion to water. Dilute emulsion should be applied using a distributor.

Alternative to the conventional anionic A3 or K3 grade emulsions is a proprietary cationic emulsion produced from soft- or hard-penetration grade bitumen. This emulsion should be applied undiluted using the distributor. Typical rates of spread should provide similar amounts of the residual modified bitumen as the conventional grades, the exact rates of application are to be decided through field trials. This alternative binder has the advantage of a tack-free surface, better physical properties and can be trafficked within 30 minutes of application. The product specification is as follows:

1. It requires no dilution. It is applied directly.
2. It is a cationic emulsion.
3. It is polymer-modified.
4. It has a residual binder of 20/30 pen.
5. It cures faster, i.e., 30 minutes to 2 hours.
6. The softening point for the residual binder is a minimum of 65°C.

Typical application rates are 0.2 to 0.65 litres/m^2 of emulsion (0.12 to 0.4 litres/m^2 of residual bitumen), depending on the texture and porosity of the surface. The optimum application rate of the solution is the maximum, which the surface will absorb without run-off. This can only be determined by field trials. Traffic can usually be allowed on the treated surface within 2 hours but this can be reduced to less than 1 hour when harder polymer-modified binders are used.

Generally, successful surface enrichment will improve the flexibility and imperviousness of old surfacing. Hence the depth of penetration of the solvents/volatiles in the fog spray into the surfacing is a critical design factor.

11.3 Crack Sealing

When cracking is not accompanied by deformation, sealing the cracks to prevent water ingress may be an effective remedial measure, at least in the short term.

Various sealing compounds may be employed. The choice depends principally upon the width of the cracks.

1. Narrow cracks (less than 2 mm in width) are difficult to seal effectively. The use of the following sealers may be envisaged:
   a. Emulsion slurry seal type I.
   b. A mixture of fluid cut-back bitumen and sand.
   c. A mixture of rubber latex and bitumen emulsion (one part in ten parts).
2. Medium cracks (2 mm – 5 mm in width) may be sealed with an emulsion slurry seal or rubber-bitumen sealing compound. The cracks should be thoroughly cleaned with compressed air before filling with the sealing compound using a pouring pot and hand squeegee or special extrusion equipment.
3. Large cracks are usually accompanied by distortion and require more extensive treatment (removal and replacement of defective areas or overlay).

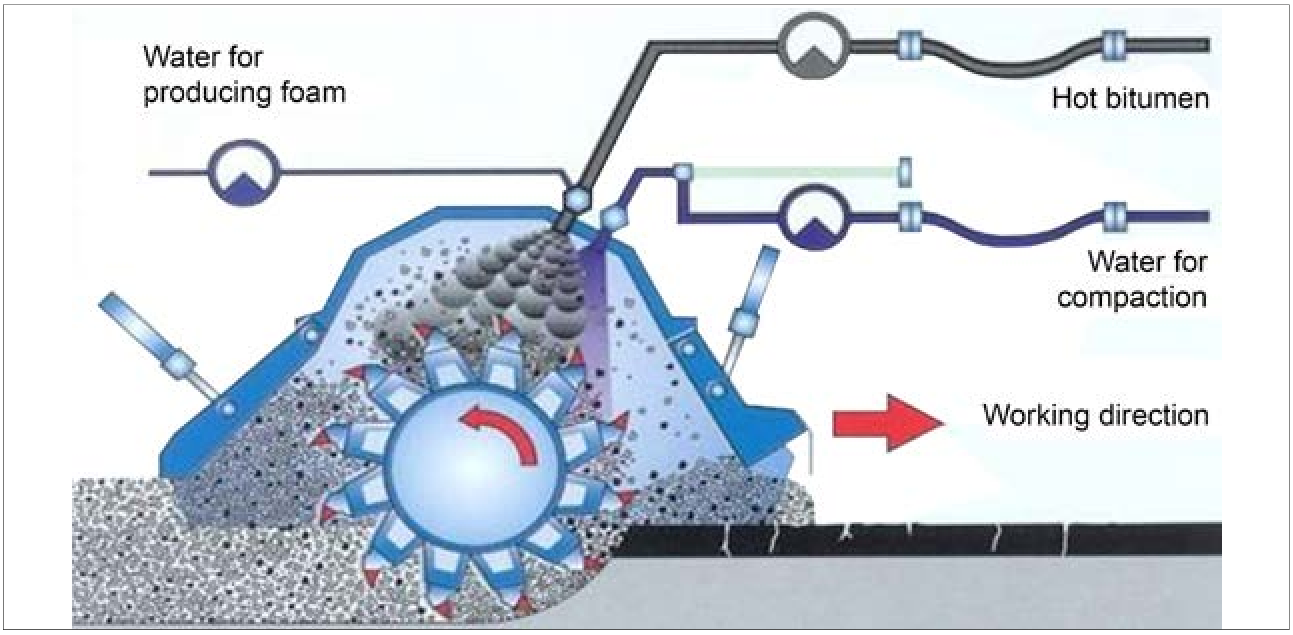
11.4 Recycling

Recycling involves the milling of old asphalt in the base or wearing course, processing and mixing with new constituents of binder and aggregate or recycling agent to soften the binder and relay as base or subbase. Where traffic volumes are low the recycled asphalt can be relayed as a wearing course provided the specifications of the wearing course are met.

11.4.1 Foamed Bitumen Recycling

This is one of the most preferred techniques for recycling asphalt using recyclers, Figure 11.1

Figure 11.1 Pavement Recycling Using Foamed Bitumen



11.4.1.1 Key Principles of Foamed Bitumen Recycling

The key principles of Foamed Bitumen Recycling are:

1. Bitumen is heated to application temperature and cold water is added at a rate of 2.5 % onto hot bitumen causing it to foam aggressively and expand by approximately 15 times.
2. Mixing is carried out immediately.
3. While the aggregate may not be fully coated, the bitumen will have a high affinity for the fine aggregate (< 0.075 mm) and form a mortar that glues the pavement material.
4. The foamed bitumen subsides within 10-20 secs as mixing is completed.
5. The foamed bitumen exhibits good dispersion properties and results in a viscoelastic pavement performance with some characteristics similar to asphalt.
6. The bitumen to be added shall be determined using laboratory-foamed bitumen pavement material recyclers.

11.4.1.2 Advantages of Foamed Bitumen Recycling

The key advantages of Foamed Bitumen Recycling are given below:

1. It produces a strong, durable and flexible pavement with viscoelastic behaviour resisting permanent plastic deformation with reduced deflections
2. Reduces maintenance requirements and cost in whole life cost terms
3. Significantly reduces or eliminates excavation volumes and the need for imported materials.
4. Minimises inconveniences to the public by speed and efficiency of the construction process.
5. Foamed bitumen recycled pavements can be trafficked immediately after construction (this is mostly required in rehabilitating urban roads).
6. It is resistant to moisture and pumping of fines due to the coating of the fines, tensile capacity and low permeability.

7. Conserves aggregate sources and it is an environmentally friendly rehabilitation option.
8. There is no risk of shrinkage cracking due to the low content of water, binder and being a flexible pavement.
9. It is a cost-effective pavement reconstruction and is significantly cheaper than deep pavement lifting.

11.4.2 Recycling Rigid Pavements

For rigid pavements, crack and sit method is used on the existing rigid pavement to turn it into a base and overlay with new concrete or asphalt. However, concrete can also be milled to the depth to which deterioration would have reached and recycled.

11.5 Inlays

Inlay or 'mill and replace' is the most preferred option of rehabilitation especially in urban areas where the elevation of final road levels should not be increased to maintain access to properties and other roads or where clearance of overpasses should not be reduced or where kerbing and storm drainage cannot be lifted.

Inlays are also applied where strengthening using overlays is not required because the pavement structure is evaluated to be adequate for future traffic. Asphalt deteriorates mostly at the surface due to environmental factors through oxidation of the binder and hardening due to loss of volatiles. Cracking generally starts from the top down. During pavement condition surveys, the depth to which cracks would have propagated is determined. The AC should be milled to this depth and replaced with new or recycled asphalt. This is the most economical way of rehabilitation.

Pavements designed to 80 MCESA become long-life pavements, and no more overlays will be required even if traffic increases beyond 80 MCESA. Interventions should only involve inlays. This is the most used approach in the UK and other jurisdictions.

For AC – Mill and replace with new asphalt of equivalent or higher strength.

For rigid pavements – mill and replace with new concrete of equivalent or higher strength or AC type I.

11.6 Reclaimed Asphalt (RA)

Mainly for AC/binder course – involves milling, reprocessing and applying granulated asphalt as base or subbase or fill or even landfill or for preparation of recycled asphalt.

11.7 Reprocessing

Mainly for granular layers – involves excavation or milling, remixing and recompacting.

11.8 Pothole Patching

Potholes need to be patched properly before applying overlays. The materials used for pothole patching should be similar to the material in the pavement layers being patched:

1. Granular materials should be used to patch potholes in granular pavements. Using of rigid or discrete elements like concrete blocks for pothole patching in flexible pavements should be avoided.
2. For deep patching similar materials should be used for each layer e.g. natural granular materials for natural subbases and bases, and GCS materials for GCS layers and asphalt materials for asphalt layers.

3. Strength enhancement may be required where adequate compaction cannot be achieved due to confined spaces or where heavier compaction equipment cannot access. Additional strength should be achieved by stabilising the materials with lime (for natural plastic soils) or cement (for natural soils and GCS) at 3 % - 5 % by mass.

11.9 Edge and Shoulder Reinstating and Improvement

Properly shaped and strong pavement edges and shoulders are essential for road safety, pavement drainage and edge restraint. The shoulder should be compacted to the same standard as the carriageway.

Pavement rehabilitation should always include the reinstatement or upgrading of weak, worn or eroded shoulders. If the edges of the pavement layers are affected by deformation or spalling they should be repaired at the same time as the rehabilitation. These works may be combined with the placing of a drainage layer.

11.10 Widening

The pavement structure for widening shall be constructed in the same manner as for a new pavement at the same location. If the existing pavement is to be overlaid, it is recommended that the widening strip be constructed up to the level of the existing surface. The whole surface is then covered with the overlay, both old pavement and widening, to ensure a neat and uniform final appearance.

Attention is drawn to the difficulty of properly laying and compacting narrow strips of materials. Special compacting equipment will be required.

It is desirable to construct widening with materials of the same type and stiffness as those forming the existing pavement to prevent longitudinal cracking at the joint.

Care shall always be taken to ensure that, after widening, the existing pavement layers and the widened strip are properly drained.

11.11 Pavement Reconstruction

Pavement reconstruction may be necessary for the following two main reasons:

1. The geometric standards of the existing alignment are inadequate.
2. The existing pavement layers are in too poor a state to be economically overlaid.

In the first case, a new alignment is necessary. In the second case, the choice of using the old alignment or a new one will be governed by the geometric standards of the existing road and the respective costs of the different solutions, (such costs should include passing traffic, drainage and structures). The compaction judgment form in Appendix C should be used for quality control.

11.11.1 Reconstruction on the Same Alignment

Reconstruction means that existing surfacing and road bases should have to be broken up and either removed or reprocessed.

The new pavement will then be redesigned using RDM 3, considering the subgrade bearing strength and the characteristics of all pavement materials, whether old or new.

Depending on how the existing pavement materials are reused, the following cases given in Table 11.1 may be identified:

Table 11.1 Reconstruction of Pavements

Item No.	Reconstruction approach	Key Considerations
1	**Removal and Replacement of the Defective Pavement Layers**	Poor quality materials, polluted crushed stone layers; disrupted bound layers, etc. should be removed and disposed of.
2	**Scarification, Reshaping and Recompacting Deformed Layers**	Flexible layers affected by extensive deformation and under-compacted may, if they are not too degraded or polluted, be reused by scarification, reshaping and recompaction.
3	**Breaking Reseating Damaged Rigid Layers**	Certain defects peculiar to rigid pavements, such as faulting or rocking or unstable slabs, render overlaying impractical. It is sometimes possible to eliminate rocking slabs by rolling with heavy rollers. The aim is to break the unstable slabs and reseat them firmly on the underlying layer.
4	**Breaking damaged rigid layers into small fragments**	Another method of eliminating rocking or unstable slabs is to break them into small fragments, with a drop hammer. Once the rigid layer has been broken, it should be graded, watered and compacted in the same manner as a crushed stone layer.
5	**Treatment of defective layers with cement and lime**	It is sometimes advantageous to treat inadequate pavement materials, by mixing then in-place with lime and cement. The construction procedures are the same as for lime or cement in materials for new roads. This type of treatment is particularly relevant to pavement materials having a high plastic index
6	**Retread process**	This process is particularly relevant to deteriorated water-bound macadam or graded crushed stone bases covered with a surface dressing. It comprises the following operations: ***a.*** Scarification of the old pavement ***b.*** Addition and mixing of new aggregate, if necessary ***c.*** Reshaping ***d.*** Application of bituminous binder and mixing in place ***e.*** Compaction Application of surface treatment Suitable binders are as follows: ***a.*** Anionic emulsion A2 and A3 ***b.*** Cationic emulsion K2 ***c.*** Medium-curing cut-back MC 250 and MC 800 Mixing may be done by a grader, but a more uniform mix will be achieved by using a pulvemixer. However, pulvemixers can be used only if there are no stones larger than 50 mm. Usually, this treatment involves the upper 100 mm – 150 mm (compacted thickness) of the old road.

11.11.2 Reconstruction on a New Alignment

The pavement shall be designed in accordance with RDM 3 for New Roads.

12 Considerations for Climate Resilience and Mitigation

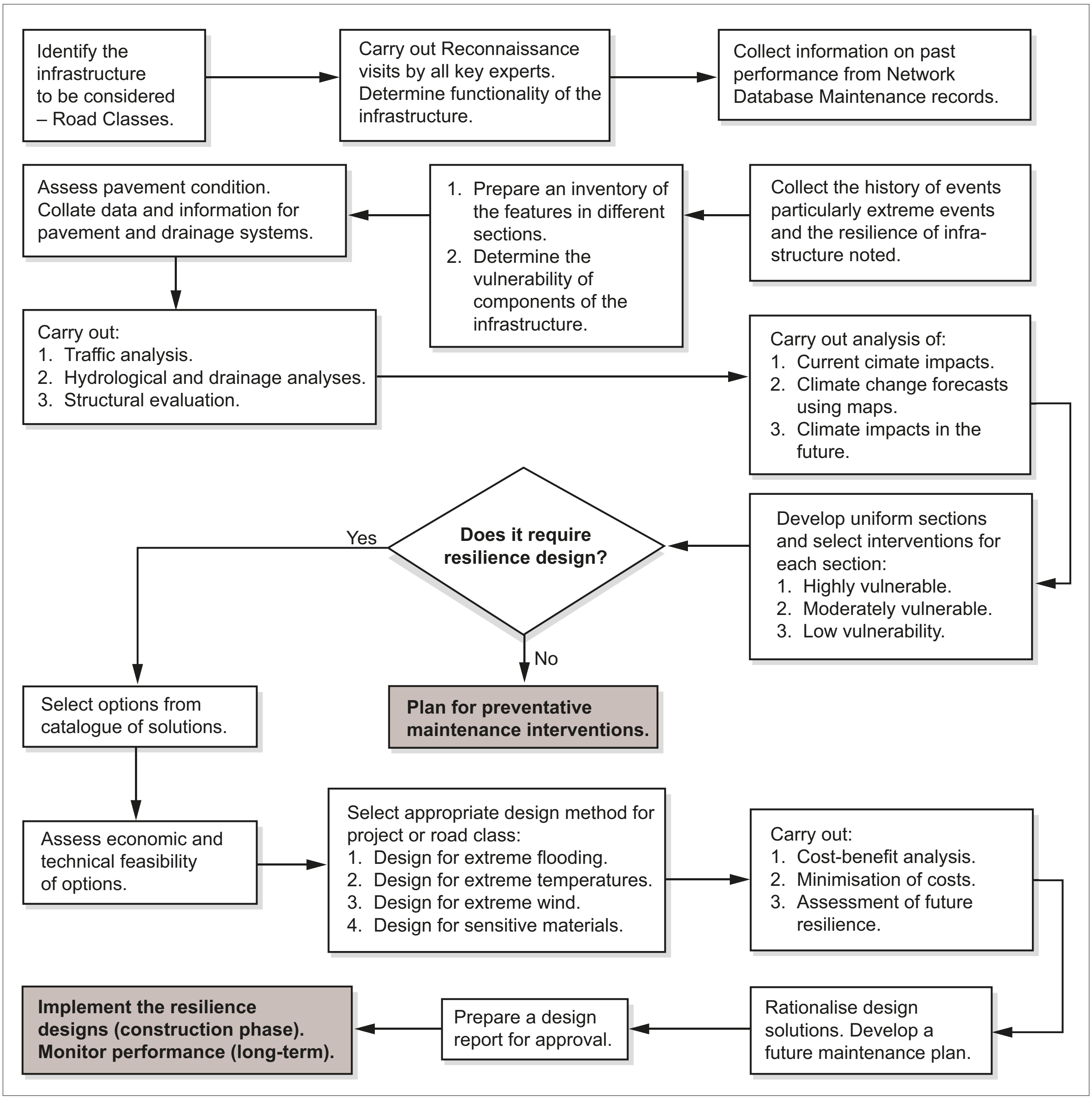
12.1 General

This chapter covers climate resilience and mitigation considerations for pavement maintenance and rehabilitation design. The key engineering elements involved in evaluating climate resilience including climate risk and vulnerability are covered in detail considering the Kenya situation. More importantly, this section provides guidance on the solutions to be considered in pavement rehabilitation design, particularly in high-climate risk areas.

Climate change mitigation covers details of the measures needed to make infrastructure projects, infrastructure maintenance and transport green (i.e., by reducing carbon footprint).

Figure 12.1 is a flowchart for assessing, evaluating, analysing, designing and implementing climate resilience in pavement maintenance and rehabilitation design. The process should be followed to ensure that the solutions are appropriate for the road class and functionality and sustainable economically and technically.

Figure 12.1 Flowchart For General Requirements In Developing Climate Resilient Infrastructure



12.2 Description of Climate Change

Climate change is a major component of road management, especially pavement maintenance and rehabilitation. The prediction models, developed over many years of performance monitoring are being affected by climate change and known limits of flooding and temperature are being exceeded.

Climate change should be properly defined to avoid ambiguities. To do this, different scenarios need to be considered. Weather patterns can change in the short, medium and long term or permanently and the following definitions should be considered carefully.

1. **Climate variation** – short-term changes to the climate e.g., where rainfall and/or temperature increases for a few years and then drops. This phenomenon may unpredictably repeat itself.
2. **Climate anomaly** – an irregular or unusual climate and unexpected phenomenon that occurs for a short period say a couple of years, e.g., arid regions receive very high rainfall, normally expected in wet regions or when temperatures may soar in temperate regions.
3. **Climate change** – a permanent shift in climatic parameters, which in the short term, can be considered irreversible.

12.3 Climate Change Factors

Climate change factors, which should be considered in road provision include:

1. Extreme temperatures - Temperatures are becoming extreme in some areas, and this is detrimental to the performance of road pavements because high temperatures cause accelerated deterioration of bituminous surfacing through rapid loss of volatiles and high rates of oxidation of the binder causing embrittlement of the surfacing. This leads to premature crack initiation, development of potholes and ultimately, premature failures of the pavement.

   Extreme temperatures can cause asphalt to soften and deform and it could happen in days or hours.

   Extreme rainfall – studies in many countries including some parts of Kenya have shown that the actual annual rainfall has not increased significantly and, in some cases, may have decreased. However, in many areas of the world including Kenya, rainfall intensity has increased with a higher prevalence of tropical depressions or cyclonic storms. This has inevitably increased the risks of damage or washaway road pavements and drainage structures.

   Additionally, extreme flooding is causing prolonged inundation of the road pavements.

   Under these extreme conditions, most road networks have become more vulnerable, and the annual cost of emergency works is high and unsustainable.

2. More frequent extreme weather conditions – are critical in the design of drainage structures and the following key aspects need to be considered:

   a. Rainfall intensity duration frequency (IDF) curves – should be reviewed regularly in light of the changing climatic conditions. IDF curves for Kenya should be developed and updated using local data. They influence rehabilitation design in determining embankment heights and the choice of pavement types and materials.

   b. Storm return periods, also referred to as flood return periods or storm frequency, should be reviewed because climate change has generally shortened the return periods of some destructive floods, making road infrastructure more vulnerable through significant increases in risks of damage and washaways.

3. Rising sea levels – Kenya has a long coastline and rising sea levels will affect coastal areas and any road pavements built along the coastline. This may also include the inundation of land where settlements have been established, including parts of towns causing prolonged inundation of pavements.

12.4 Climate Risks and Vulnerability of Pavements

Climate risk is a key design consideration in the rehabilitation of pavements. The road network consists of sections under high climate risk and other parts with low climate risk.

Understanding climate risk for pavements allows the engineer to design solutions to minimise negative impacts on its performance. These solutions will enhance pavement resilience. In this Chapter, details of challenges and recommendations are covered to ensure that the pavements are made more climate-resilient following rehabilitation.

The determination of the level of climate risk is an important part of the investigations and reviews, which should be undertaken throughout the project cycle.

1. During project planning – rehabilitation design should involve retrofitting climate resilience in areas of high climate risk.
2. Pavement rehabilitation design – should include assessing and retrofitting climate resilience during rehabilitation design. The solutions should be sustainable in cost, technical appropriateness, and robustness.
3. Pavement construction – constructability and costs should be considered at this stage to ensure that the solutions are carried through for the benefits of the extra investments to be realised.
4. Research – the climate is evolving to defeat the efforts of the design engineers and therefore continual research and trials should be included in rehabilitation schemes.

12.4.1 Climate Risk Considerations for Pavement Resilience Design

Climate risk is determined considering the following.

1. Severity of impacts of climate change – It is important at the stage of planning and design that a thorough assessment of the level of climate risk of the project areas is known. This is important because investments should be matched with the anticipated severity of the impacts of climate change. For pavement maintenance and rehabilitation, the planner and design engineer should consider the following:
   a. Increase in the frequency and intensity of rainfall - Determine the increase in the frequency of flooding and inundation of pavements and road foundations for longer periods. Discover how this impacts the design, material selection and performance predictions of pavements.

      Relevant information can be obtained from:

      i. Kenya flood risk maps, Figure 12.2.

      ii. Climate change projection maps ideally covering 30 years to 50 years.

      iii. Kenya climatic maps – general climatic maps, which have not been adjusted based on climate change. The design engineer should use the maps to evaluate historical pavement performance under normal conditions.

      iv. Kenya topographic and geological maps – obtain the maps from relevant authorities. These data and information will influence the rehabilitation design of the pavement where, in severe cases, the structure and materials for the pavement should be designed for inundation. The design should allow the pavement to retain its functionality when submerged. It, therefore, means that materials and pavement structures should specifically be selected and designed to suit severe conditions.

      v. Kenya groundwater maps – a significant percentage of pavement failures or poor performance is caused by prevailing groundwater conditions or significant changes to these conditions, that impact the resilience of the pavement and its materials. Prior knowledge of this, will influence/inform the pavement condition investigations/ surveys given in RDM 5.1.

1 2 3 4 5 6 7 8 9 10 11 12 Considerations for Climate Resilience & Mitigation

   vi. Flood history of the road and project area – This is a critical component of the assessment of the potential future performance of the pavement regardless of the designs. Frequent flooding can reduce the life of the pavement significantly.

b. Determination of potential impacts of extreme flooding leading to damage to the pavements and road foundations. The following are key design considerations:

   i. Extreme force – Extreme flooding usually causes strong hydraulic forces on the pavement which may cause the surfacing to break up leading to severe scour of the bases and subbases as well as the subgrade. Resistance to uplifting and scour should be considered in the design especially where resilience should be retrofitted onto the existing pavements, see the catalogue of solutions given in Section 12.5.

   ii. High hydraulic head during inundation – Forces water to percolate into the pavement and layer interfaces causing delamination and debonding. It also causes saturation if the flood persists for long periods and sensitive materials can become too weak. Fluidity of the road foundation can become apparent, even for S2 subgrade (or foundation class 1).

c. Increase in temperature – Climate change causes gradual as well as extreme increases in temperatures and this has design consequences:

   i. An increase in temperature implies a significant reduction in strength for bituminous materials, particularly AC, DBM and other bituminous seals. In the mechanistic design method detailed in Section 9.6, the moduli and deflections of the bituminous layers are normalised/standardised to 20 °C for temperate and subtropical regions and 30 °C to 35 °C for tropical climates. This standard may need to be adjusted as temperatures soar. The predicted performance will be lower than anticipated when temperatures increase substantially. A significant reduction in in-situ moduli may exacerbate plastic deformation leading to premature failures. For high-risk temperate and subtropical regions use 30 °C to 35 °C and for tropical use 40 °C to 45 °C to normalise deflections and moduli when considering excessively high temperatures.

   ii. Extreme temperature events are more serious. Under extreme temperature events, whole roads can deform to exceed failure criteria for the materials (rut depth >20 mm) in hours or days. In this case, the choice of overlays and their thicknesses need to be designed to minimise inevitable deformation by compensating the thickness of AC and DBM with layers of strong non-deformable materials, such as GCS, macadam bases, Telford bases, and cement or lime stabilised bases. Concrete overlays should also be considered, as well as heat-resistant binders, like polymer-modified binders (PMB) and fibre mastic asphalt (FMA). Also, consider using open-graded bituminous mixes that are rut resistant, and apply a seal on top to protect it against UV radiation and loss of volatiles.

2. Climate change projections are generally for 20 to 50 years on pavements. The design engineer needs to take this into account. Some of the important design considerations are given below:

   a. Climate resilience generally results in a significant increase in the capital costs of a project. The biggest challenge in rehabilitation design is determining how much retrofitting for climate resilience should be carried out and at what cost. This should be balanced with the consequences of do-nothing or do-little scenarios and their related costs.

   b. For low-volume rural and urban roads, climate change projections of 30-40 years would suffice.

   c. For high-volume roads climate projections of 50 – 70 years would be adequate.

   d. Climate projections greater than 70 years would be required for major bridges but this is not appropriate for pavements because of cost implications.

3. Potential for extreme weather conditions and anticipated recurrence – refer to the Hydrology and Drainage Design, RDM 2, for predictions of recurrence or flood return periods of extreme weather conditions. The following are important design considerations:

Determination of Annual Exceedance Probability (refer to Meteorological data and RDM 2) – This approach indicates how often extreme weather conditions are likely to occur to guide the design engineer on how much retrofitting resilience is required. These relationships can be developed using local data. The climate variable should be developed using the percentage of the frequency by which the annual rainfall and temperature averages are exceeded.

In high climate risk areas:

a. Design for overtopping of the pavement.

b. Design for long periods of inundation.

c. Design for the pavement to retain its functionality when submerged.

d. Design for appropriate internal pavement drainage.

e. Design for heat and rut-resistant AC and DBM.

4. The nature of risks – the risks should be considered in two categories i.e., risk to the infrastructure, and risk to the communities and road users.

   a. Risk to the infrastructure:

      i. Damage to the pavement – water currents with debris ripping off the surfacing, pavement layers and the road foundation. This is highly dependent on the erodibility of the pavement materials and the hydrodynamics around the pavement structure. The design engineer should consider this when selecting the materials and pavement structure.

         Also, protection of the pavement from scour should be paramount, but most importantly the type of pavement makes the biggest difference. In very severe situations rigid pavements should be the preferred choice.

      ii. Weakening of the pavement layers through the development of pore pressures within the pavement structure which could cause the pavement to disintegrate under load – Prolonged flooding of the layers can lead to the liquification of the fine and clayey subgrade or road foundation leading to premature failures, typically deformation, cracking and fracturing.

      iii. Pavements made of asphalt concrete tend to deform rapidly under load when temperatures are extremely high. Air temperature above 40 °C for prolonged periods is detrimental to the integrity of asphalt. Heat and rut resistance will be critical design considerations.

5. The distribution of high-risk areas – the distribution of high and low climate risk areas is an important consideration for the designer depending on the location of the project area. Either way, it will influence the design standards to be used.

   a. High flood-risk areas of Kenya are given in Figure 12.2

   b. High temperature risk areas are given in Figure 12.3.

6. Risks to communities and commerce served by the road infrastructure – Climate risk should be considered for infrastructure and the public.

   a. Accessibility – this is the major risk to communities and road users. Once accessibility is cut through damage to the pavements, restoration of access is usually difficult and can take a long time to re-establish. More rapid methods and quick-setting materials should be used, in rehabilitation design.

   b. Change in land use – changes in land use increase risks to the infrastructure, which in turn increases the risks to the communities and road users. The design engineer should review plans for future land use to ensure its impact can be established and catered for in the rehabilitation design. Significant changes in land use may increase high flood levels in the drainage system, causing the upper layers to be reached, overtopped or breached.

Figure 12.2 Flood Risk Areas of Kenya

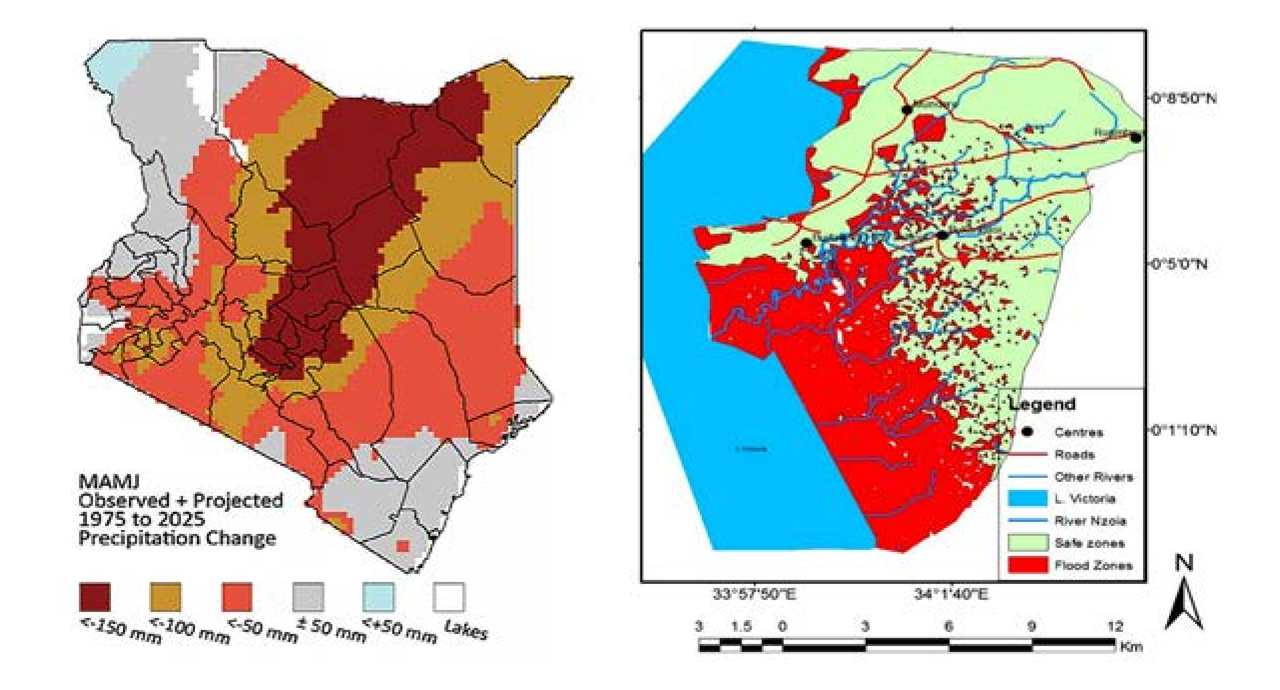
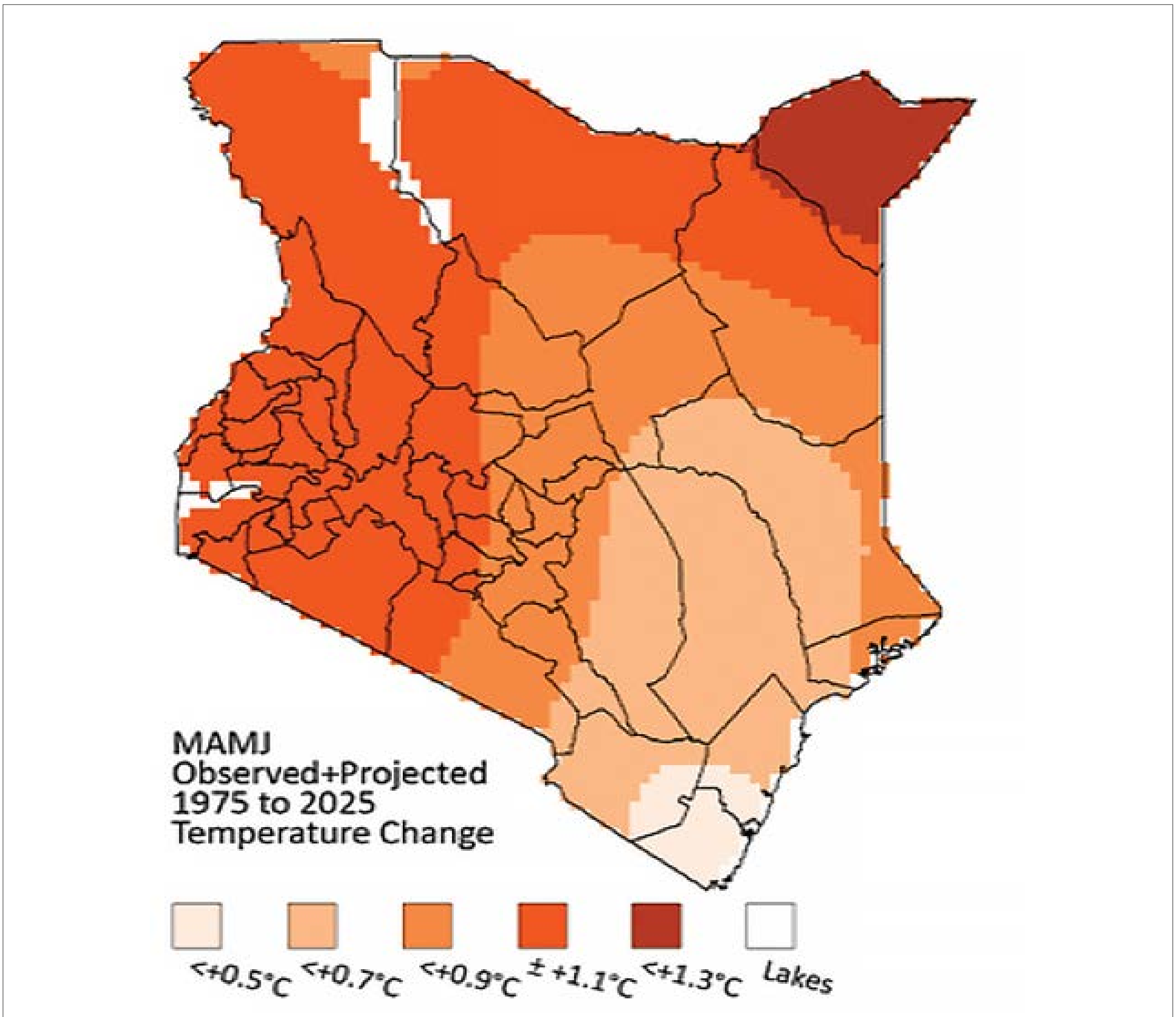


Figure 12.3 Temperature Risk Areas of Kenya



c. Road closures – these are inevitable in the case of extreme flooding where inundation is likely to occur. The pavement layers may weaken significantly, and the road foundation/subgrade and subbases are more likely to be affected. If the reduction of the strength of the materials in the soaked condition is more than 50% then the probability of premature failure will be high. The design engineer should consider carrying out a sensitivity analysis of the behaviour of the different pavement materials under submerged conditions for longer periods, Figure 12.6. Road closure should be included in maintenance plans, and should be provided in the rehabilitation design report. Where there are no alternative routes, the design should be more climate resilient regardless of the extra costs.

7. High cost of damage to infrastructure, especially pavements – most of the repairs are usually through emergency works, which tend to be much more expensive than normal projects. Other costs include loss of access to amenities, health centres and sources of supplies for the communities. The design engineer should aggregate these potential costs to justify a higher level of intervention and a bigger budget for climate resilience.

8. Delay in the restoration of pavements and drainage structures – pavements take long to build. The use of locally available materials is a key consideration in rehabilitation design. Coarse materials, where available, would be the best option i.e., nominal maximum aggregate size >37.5 mm and <70 mm. The designs should ensure climate resilience in anticipation of an early return of extreme weather events or the perpetuation of adverse conditions.

9. Premature failures or poor performance of pavements – negative impacts of climate change tend to cause premature failures or general poor performance of pavements. Knowledge of past performance of pavements existing in the project vicinity is required to guide the designer on the level of retrofitting climate resilience required.

10. High cost of retrofitting climate resilience during rehabilitation of pavements – preventative or enhanced maintenance is required to retrofit climate resilience on existing infrastructure considering the level of risk and vulnerability. A cost-benefit analysis should be carried out to determine the most cost-effective options.

11. Lack of accurate data on climate change and its impacts on pavements – data on climate change are usually not readily available. The design engineer should coordinate with maintenance sections of road authorities, other entities responsible for disaster preparedness and, above all, the Meteorological Authority to collect any relevant data on the performance of the roads and the corresponding influencing factors of climate and traffic.

12. Lack of sustainable solutions to counter impacts of climate change – sustainable solutions stem from robust research and engineering practice. The design engineer should develop solutions that are economical and durable. Solutions that may be considered are detailed in Section 12.5.

13. Lack of appropriate design standards and specifications for resilient pavements – most national and international standards do not incorporate design for resilience. This is a serious handicap, but the engineer should study the risks carefully and develop engineering solutions to minimise climate risks.

12.4.2 Climate Vulnerability Considerations

Vulnerability stems from a lack of resilience against an adverse condition. Design for resilience involves mitigating vulnerability. It is therefore necessary to understand the vulnerability of road pavement.

1. Lack of alternative routes – this makes both the pavements and the communities vulnerable in the event of an extreme weather event. Closures should be considered during and after inundation. Otherwise, the pavement should be designed to overcome extreme flooding and/or extreme temperatures and retain functionality when such events occur.

2. Pavements in high climate risk areas – use maps that show the vulnerability of the road networks and communities to determine the level of vulnerability to consider in the rehabilitation design.
3. Pavements made of climate-sensitive materials – review the structure and materials making up the pavement under design. From the investigation of materials, prepare a schedule of the vulnerability of the materials used to build each layer and the subgrade. Determine further strengthening of the pavement.
4. Condition of pavements in high climate risk areas – pavements, in a state of disrepair are more vulnerable to climate change impacts and extreme weather events. A future maintenance plan should be developed for the rehabilitation design adopted to set the level of service necessary for the road to sustain the resilience required for the high-climate risk area. Backlog pavement maintenance makes roads more vulnerable.
5. Adopting temperate climate pavement design standards – standards obtained from temperate climatic conditions are erroneously adopted in the tropical climate making pavements vulnerable to climate change impacts. Such standards should be adapted to Kenya's conditions using empirical evidence from Kenya before they are used.
6. Statutory changes in material specifications – there are statutory changes to the material specifications such as bitumen, which have reduced their quality and performance in service. This leads to poor pavement performance or premature failures, e.g., bitumen is currently required to have ≤ 2 % sulphur. This makes binders less resilient and results in premature failures or poor performance of bituminous materials.
7. Poor quality of construction – road pavements that are poorly constructed are more vulnerable to climate change impacts. They are easily washed away, and the pavement layers deteriorate rapidly under traffic loading, but predominantly due to environmental factors. Overheating binders exacerbates ageing during construction as well as the rate of oxidation and embrittlement of binders, in-service, causing premature cracking.
8. Lack of appropriate protection works – protection works are key to the resilience of the pavement in extreme weather conditions of flooding and high temperatures. This may include protecting AC from UV radiation using fog sprays or armouring pavements against floods such as armouring side slopes and armouring of bases Figure 12.4 and Figure 12.5.
9. Lack of climate resilience audit for rehabilitation projects – implementation of climate resilience needs the necessary checks to be put in place lest they be ignored or overlooked in rehabilitation design. The lack of climate resilience audits hampers implementation even for high-value projects.

12.5 Catalogue of Solutions for Climate Resilient Design

12.5.1 General

This section provides a catalogue of climate resilience risks or challenges faced by engineers in the provision of infrastructure and guidance for mitigating them through engineering solutions and other interventions. This is particularly important because often there is coverage of climate change and its impacts but very little in terms of solutions.

While solutions are much more difficult to develop because a lot of long-term research is required which very few organisations are willing to undertake. Solutions should be tailored to the class of roads and the value of the project as discussed above. No one size fits all and the guidance below may need to be modified or adapted to be appropriate for the situation risk to ensure sustainable mitigation.

Table 12.1 List of Catalogue of Climate Resilience Solutions

Item No.	Category	Catalogue Nomenclature	Scope
1	Climate Change Evaluation	CCC	1. Risk mapping 2. Vulnerability mapping 3. Risk level
2	Pavements and Materials Resilience	CPM	1. Subgrade materials 2. Base and subbase materials 3. Surfacing and wearing courses
3	Mitigation	CMM	1. Reducing the carbon footprint of road-building materials 2. Transportation 3. Construction process

The catalogues of Climate Resilience Solutions are divided into 5 categories in Table 12.1.

12.5.2 Catalogue CCC1 Climate Risk Mapping

12.5.2.1 Challenges/Risks:

1. Climate risk is a key consideration for evaluating and designing pavements, particularly for existing infrastructure. The road network consists of sections under high climate risk and other parts with low climate risk. The evaluation will provide the engineers with options for interventions in sections of varying risk levels. Inadequate evaluation will lead to poor designs, recurring rapid deterioration and premature pavement failures.

   Climate risk evaluation is a major engineering challenge requiring experience and robust data to resolve.

2. Determination of the level of risk will help to determine the challenges and engineering solution options available to consider. Additionally, it would help to determine the level of investment required and the appropriate cost-benefit analysis.

   The level of risk is critical in designing solutions to mitigate the impacts of climate change.

12.5.2.2 Solution

Climate Change Projections for Design

Climate change projections are an essential part of resilience design. There are several components to it.

1. Climate change is a key study on its own. The data are represented in maps and graphically for interpretation. It has two components:
   a. Prediction of changes in rainfall and temperatures.
   b. Predictions of extreme rainfall intensities, and periods of extreme temperatures in number of days for each year.
2. Climate change projection periods are generally 20 – 50 years for pavements. The design engineer needs to take this into account. Some of the important design considerations are given below.
   a. Climate resilience generally results in a significant increase in the capital costs of a project. The biggest challenge in rehabilitation design is determining how much retrofitting for climate resilience should be carried out, and at what cost. This should be balanced with the consequences of 'do-nothing' or 'do-little' scenarios and their related costs.
   b. For low-volume rural and urban roads, climate projections of 30 – 40 years would suffice.
   c. For high-volume roads climate projections of 50 years – 70 years would be adequate.

12.5.3 Catalogue CCC2 – Vulnerability Evaluation

12.5.3.1 Challenge

Vulnerability stems from a lack of resilience against an adverse condition. Design for resilience involves mitigating vulnerability. It is therefore necessary to understand the vulnerability of road pavements.

Determination of infrastructure vulnerability involves a lot of parameters, some of which are more significant than others and this is a key engineering challenge.

12.5.3.2 Solution

Vulnerability evaluation involves:

1. Creating an inventory of the infrastructure listing the different components and their location in chainages:
   a. Types of pavements and their locations recording the chainages from start to finish.
   b. Structures – bridges (large and small), culverts (types and sizes), drains (lined or unlined), stormwater drainage features (types, purpose, location
   c. Carrying out a detailed condition survey of the infrastructure.

Note: *deteriorated infrastructure components tend to be more vulnerable to impacts of climate change including extreme weather events.*

2. Combine the data and information from the risk maps and the inventory as well as the condition of the components of the infrastructure to determine the level of infrastructure vulnerability. Prepare:
   a. A line diagram of the route indicating the location of the different infrastructure components.
   b. Assign the level of vulnerability based on Table 12.2.

Table 12.2 Criteria for the Level of Vulnerability

Item No.	Level of Vulnerability	Range	Climate Vulnerability Evaluation Criteria
1	Very high	0.7 – 1.0	*1.* High-risk zone based on maps, and *2.* The infrastructure is in poor condition, and *3.* Inadequate protection
2	High	0.5 – 0.7	*1.* High-risk zone based on maps, and *2.* The infrastructure is in fair condition, and *3.* Inadequate protection
3	Moderate	0.2 – 0.5	*1.* Moderate risk zone based on maps, and *2.* The infrastructure is in good condition, and *3.* Some protection is provided
4	Low	< 0.2	*1.* Low-risk zone based on maps, and *2.* The infrastructure component is in good condition, and *3.* Adequate protection is provided

12.5.4 Catalogue CCC3 – Level of Risk

12.5.4.1 Challenge

The level of risk (Table 12.3) determines the level of investment required to mitigate the risk adequately.

Engineering determination of climate risk is complex because it includes many variables and assumptions.

12.5.4.2 Solution

There are many ways of determining the level of risk, but in this manual, it is recommended to calculate using the level of vulnerability and probability of exceedance of design parameters, including temperature, rainfall intensity, and flooding.

The probability of exceedance can simply be developed from historical data on:

1. Flood levels in rivers and streams.
2. Rainfall intensity.
3. Maximum annual rainfall.
4. Maximum temperatures and their duration.
5. Maximum wind speed for high bridges.

These are the design parameters used in the design of the pavements, and drainage structures and systems. A thorough review of the designs and as-built data is important at this stage.

The historical data is then analysed to determine the number of times that the design parameters have been exceeded and expressed as a percentage of exceedance (P_e). Hence:

$$P_e = \frac{occurrence\ of\ exceedance}{total\ occurrence} \times 100$$

The next stage is to determine the change in climate factors of:

1. Rainfall intensity-duration-frequency.
2. Temperature intensity-duration-frequency.
3. Wind intensity-duration-frequency (for high bridges and cable stay footbridges).

For rainfall intensity, use the IDF curves developed for local rainfall conditions by the Meteorological Authority.

For temperatures, use the historical data to determine the duration of maximum temperatures i.e., the number of days of occurrence in a year. The intensity and duration of maximum temperatures are bound to increase significantly with climate change.

For wind, use historical data to determine the duration of annual maximum wind speeds for intensity duration and frequency. This is also bound to increase significantly with climate change.

For temperature :

1. Maximum temperatures reached.
2. Duration in days per year that design temperature is exceeded and if these have increased due to climate change.

$$Level\ of\ Risk,\ R = V \times P_e$$

Equation 12.1

Where,

R = Level of risk, %

V = Vulnerability, ranging from 0.1 - 1

P_e = Probability of exceedance, %.

Percentage risk is categorised in Table 12.3,

Table 12.3 Categories of Risk for Determination of Intervention Levels

Item No.	Level of Vulnerability	Range	Climate Vulnerability Evaluation Criteria
1	Very high	≥ 70	1. High-risk zone based on maps, and 2. The infrastructure is in poor condition, and 3. There is inadequate protection
2	High	50 - 70	1. High-risk zone based on maps, and 2. The infrastructure is in fair condition, and 3. There is inadequate protection.
3	Moderate	20 - 50	1. Moderate risk zone based on maps, and 2. The infrastructure is in good condition, and 3. Some protection is provided.
4	Low	< 20	1. Moderate risk zone based on maps, and 2. Infrastructure is in good condition, and 3. Some protection is provided.

12.5.5 Catalogue CPM1 – Subgrade Materials

12.5.5.1 Challenge

Subgrade materials can be in-situ or imported, sometimes called selected materials. The subgrade layer is formed using in-situ materials and compacted to achieve the minimum CBR requirements for the design. Most subgrades tend to be fine and plastic, but sandy subgrades are generally non-plastic and others can be coarse with different levels of plasticity.

The subgrade materials that are most sensitive to climate change impacts are problematic soils, namely:

1. Expansive clays – inundation for long periods causes the clays to liquefy. Moisture variations cause the clays to heave resulting in longitudinal and transverse cracking and severe deformation.
2. Erodible soils – this normally results in the collapse of pavements.
3. Collapsible soils – these are soils that form a macrostructure with weak bonds which upon ingress of moisture and loading collapse causing cavities under the road in which case the pavement may deform or collapse.
4. Saline soils – caused by the inundation of coastal areas with salty sea water or soils off the coast with high salt content. High salt content caused the pavement surface to blister and disintegrate.
5. Degradable soils – materials that degrade rapidly when placed in a pavement. Basaltic soils degrade to form plastic fines, thus affecting the performance of the road pavements.

The engineering challenge is how to make problematic subgrade materials withstand the impacts of climate change.

12.5.5.2 Solutions

1. For expansive materials – design for treatment of expansive materials referred to as TE treatment. It involves cutting waste to a depth of 600 mm and backfilling in layers of 200 mm with fines and slightly plastic materials. After excavation, the surface should be sprinkled with water to keep the clay moist and then covered immediately to prevent moisture loss. The backfill seals the clay from moisture variation to maintain equilibrium moisture content preventing heaving of the clay.
2. Erodible soils – should generally be avoided, but if it is not possible, armouring or cladding or vegetation cover or prevention of strong water currents will minimise erosion. In areas of high climate risk stabilisation with hydraulic binders or bitumen emulsion should be considered.
3. Collapsible soils – use heavy vibratory compactors or impact rollers to collapse the soil structures before constructing a pavement on top. Collapsible soils can be identified by the prevalence of sinkholes in the surrounding land.
4. Saline soils – apply a coarse-grained layer on the saline material. This will stabilise it and prevent blisters from affecting the rest of the pavement. In high climate risk areas, cut-to-waster and replace with non-saline materials.
5. Degradable soils – in high-climate risk areas, select coarse materials that minimise the effects of the gradual increase of the content of plastic materials within the subgrade layer.

12.5.6 Catalogue CPM2 – Base and Subbase Materials

12.5.6.1 Challenge

Bases carry most of the traffic loading and less so for subbases. Weak bases degrade through cracking and deformation. The bases and subbases should be distinguished using their material characteristics. They can be classified as:

1. Coarse or fine.
2. Non-paedogenic and paedogenic.
3. Plastic or slightly plastic or non-plastic.
4. Unbound – granular.
5. Bound – stabilised with cement, lime, and bitumen.
6. Conventional – these are considered to be mostly specified in the standards and the characteristics defining them are explicitly given.
7. Non-conventional - materials that are not commonly specified for bases and subbases and are normally considered in design as out of specification.
8. Flexible – pavements made of granular materials, or materials modified with cement or lime, and may have an asphalt wearing course with thickness ≤ 50 mm.
9. Semi-rigid – are pavements with thick asphalt > 50 mm, such as dense bituminous macadam (DBM) and dense emulsion macadam (DEM).
10. Rigid pavements – concrete pavement, which can be unreinforced slabs, jointed reinforced, or continuously reinforced. These can be composite:
    a. Concrete base with asphalt surfacing
    b. Asphalt with concrete surfacing.

Selection of the most appropriate option of base or subbase for the different climate risk categories and making the pavement layers resilient to the impacts of climate change is a big engineering challenge.

12.5.6.2 Solution

For high-climate risk areas consider the following:

Extreme rainfall and flooding negatively impact the integrity of the bases and their performance.

1. For natural bases and subbases:
   a. Carry out moisture sensitivity analysis, see Figure 12.6.
   b. The material must be self-draining – check the material permeability in the laboratory. This is to avoid the build-up of pore pressures.
2. Use very coarse gravels.
   a. Conventional materials – graded crushed stone (crushed stone base), 37.5 mm nominal size.
   b. Unconventional material – **macadam base** material, 70 mm nominal size or **Telford base** materials, 150 mm nominal size.
3. Armoured base
   a. Unconventional thin armouring – fine bases armoured with a single layer of 30 mm – 50 mm aggregate hammered/compacted into the base to refusal using very heavy vibratory steel rollers operating at high amplitude, till refusal, Figure 12.4.
   b. Unconventional thick armouring – fine bases armoured with a thick layer of aggregate with the bottom part of the layer partially mixed with the fine base material. The base is scarified to a depth of 50 mm and the layer of stone is applied and mixed with the base and hammered/compacted as in **1.b.** above. An additional layer is then applied on top and compacted heavily, Figure 12.5.

Figure 12.4 Typical section – Armoured Pavement/Base and Amalgamated Surfacing for Climate Resilience

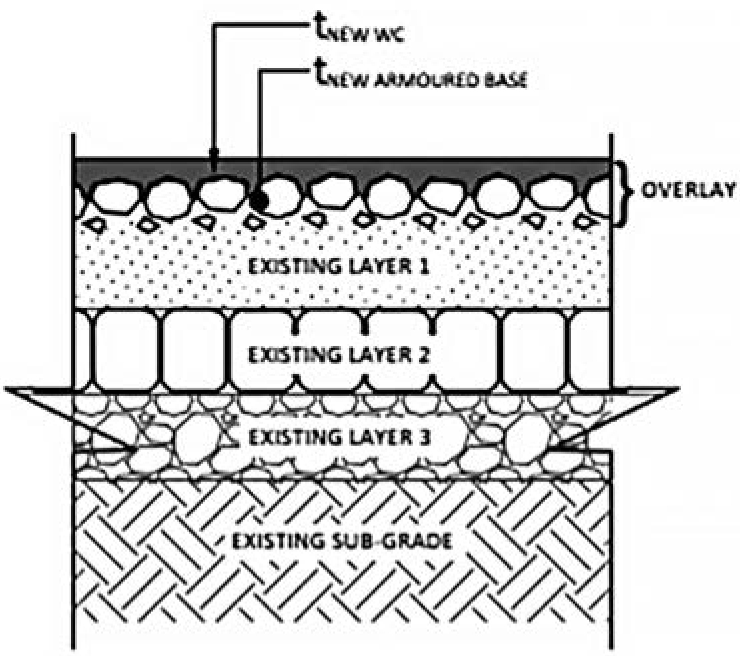
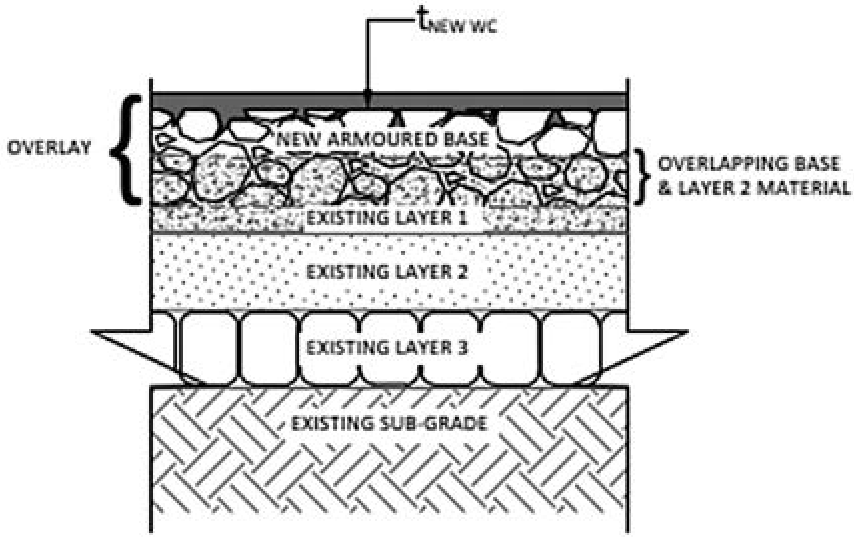


Figure 12.5 Typical Detail of Thick Armoured Bases and Amalgamated Surfacings for Climate Resilience

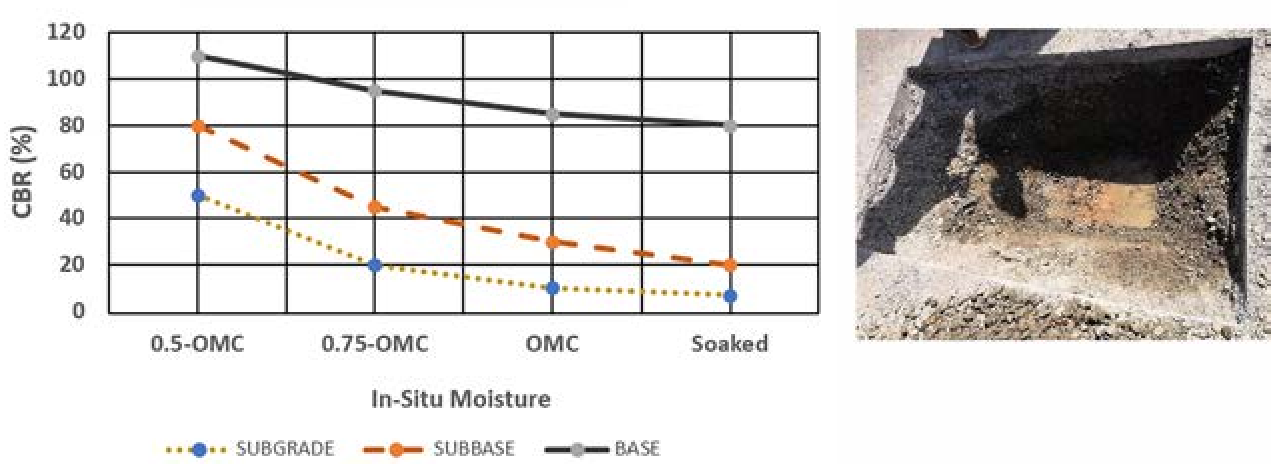


4. Rigid pavements
    a. For LVRSs – design a 100 mm to 150 mm concrete base of unreinforced slabs on a granular subbase. Wire mesh reinforcement if required should be placed at one-third of the depth of the concrete slab thickness from the top for crack prevention.
    b. For HVRs – design a concrete base of 150 mm – 200 mm concrete on top of a cement or lime or bitumen stabilised subbase. This can be dowel jointed slabs or continuously reinforced concrete with reinforcement of 16 mm bars at 200 mm to 400 mm spacing.

For pavements that are prone to flooding and inundation for long periods consider the following:

1. Significant weakening of the pavement layers and subgrade – consider the following:
    a. Use traffic damaging factors higher than 4.5, recommended values are 5-7. Exponent 7 would be for extremely vulnerable pavements.
    b. For subgrade strength carry out a moisture sensitivity test, see the example in Figure 12.6.

Figure 12.6 Moisture Sensitivity Test



1
2
3
4
5
6
7
8
9
10
11
12
Considerations for Climate Resilience & Mitigation

i. Determine the CBR at different moisture contents and the lowest value when the material is soaked or saturated. Saturation is assumed to be reached after 8 – 10 days of soaking. The results will indicate the moisture sensitivity of the material.

ii. Determine the critical strain of the subgrade and compare it with the critical strain obtained through deflections or assumed CBR at equilibrium moisture conditions in the field and CBR after the normal 4-day soaking. This will give the potential for the deformation of the subgrade under repeated heavy loading.

iii. Check the lowest CBR obtained in the sensitivity test analysis and compare it with the design-soaked CBR. If the difference is significant (> 50 %), the material may deform under load during prolonged extreme flooding or inundation.

c. Carry out similar sensitivity tests for natural bases and subbases including emulsion-treated bases or bases stabilised with foamed bitumen, cement or lime. Any drop of strength of 50% or more implies a high vulnerability of the materials and the pavement.

12.5.7 Catalogue CPM3 – Surfacing/Wearing Course

12.5.7.1 Challenge

Climatic condition is a key element in the design of surfacings especially bituminous and concrete pavements. The design standards for normal temperature are well-developed and well-understood by design engineers.

The rapid increase in temperature being caused by climate change and the ever-increasing frequency of extreme temperatures is causing serious challenges for engineers. The exceedance of standard limits renders some design methods and specifications inadequate to provide the required resilience.

Environmental laws are restricting the use of some of the conventional products developed to cope with high temperatures and there are no alternatives, in some cases. For example, the sulphur content in bitumen has been reduced from 4 % to 2 % with a desirable maximum limit of 0.5 %. Sulphur greatly increases the performance and durability of bitumen, hence with the reduction of its content the quality of bitumen has significantly reduced. This causes poor performance, or premature failures of bituminous pavements and surfacings.

12.5.7.2 Solution

1. Resilient bituminous surfacings – design considerations for high and extreme temperatures

   Increase in temperature – Climate change causes gradual as well as extreme increases in temperatures and this has design consequences:

   a. Rapid increase in temperature – several solutions to be considered by the engineer include:

      i. Use heat-resistant binders – including polymer-modified binders, rubberised binders (could be from waste tyres), binders modified with plastic (could be waste plastics).

      ii. Use binder with less volatile solvents – Use oil or diesel-based cutback binders, which deteriorate less rapidly in conditions of high temperatures.

      iii. Use Dense Bituminous Macadam (DBM) – which is much less deformable at high temperatures.

      iv. Use Dense Emulsion Macadam – which is less deformable and has a low carbon footprint.

      v. Use fibre mastic asphalt – which is more resilient to cracking and has improved the longevity of the asphalt or surfacing.

      vi. Apply thin bituminous surfacings on bases that exhibit low deflections under loading.

vii. Apply a minimum thickness of AC of 75 mm to allow for future inlays as deterioration or cracking usually starts from the top down. The concept is to mill deteriorated AC to the depth where cracking would have reached from the top and replace it with new and stronger asphalt.

viii. Construct amalgamated surfacings – these are surfacings that do not have a plane interface with the base. Part of the base protrudes into the surfacing and part of the surfacing intrudes in the base. Such surfacings are highly durable and resilient.

ix. In the Mechanistic Design Method, the strength measured in moduli of elasticity (E-Moduli), or deflections are normalised/standardised to 20 °C for temperate and subtropical regions and 30 °C to 35 °C for tropical climates. This standard may need to be adjusted to 40 °C – 45 °C as temperatures increase to accommodate higher operating temperatures due to climate change. The predicted performance will then be lower than anticipated when temperatures increase substantially. A significant reduction in in-situ moduli may exacerbate plastic deformation leading to premature failures.

2. Extreme temperature events – Under extreme temperature events, whole roads can deform to exceed failure criteria for the materials (rut depth >20 mm) in hours or days. Heat-resistant binders and less deformable materials should be considered.

   a. For bases e.g. GCS – the base should have a rough finish such that part of the base protrudes into the AC and part of the AC intrudes the base resulting in amalgamated surfacing, which is more resilient.

   b. For double surface dressing – the size of the aggregate of the second layer (on top of the first layer) should be < 0.5 the size of the aggregate of the first layer. This causes the amalgamation resulting in no plane interface between the 2 layers.

   c. Thin medium-graded AC (max. 50 mm thick) on macadam-based – this combination prevents deformation of AC at high temperatures which counters the negative impacts of extreme temperatures on asphalt. The macadam base exhibits very low deflection hence AC will be less susceptible to cracking, which minimises deterioration even at high temperatures

   d. Thin medium graded AC on Telford bases – this combination behaves as stated in 2(c) above but with better performance and greater resilience against climate change impacts of extreme rain/flooding and extreme temperatures.

   e. Double surface dressing or coarse slurry on Macadam and Telford bases – this surfacing tends to be highly durable because of the very low deflections of the pavement under load. At old age surfacing will crack but not disintegrate.

   f. Cape seal on macadam and Telford bases – Cape seal is a single surface dressing with a slurry seal on top. This combination behaves as in **2.c**. above but with better performance and resilience against the impacts of climate change.

   g. Amalgamated penetration macadam – a strong and stable surfacing. It involves the application of a prime and tack coat of emulsion at 1.0 L/m^2, followed by the application of a large aggregate of 30 mm – 50 mm, rolling. A finer aggregate of the nominal maximum size of 13 mm is then applied on top. This is followed by compaction by vibrating the fine aggregate to wedge in the interspace of the large aggregate. At this stage, the aggregate matrix is stable enough to be trafficked for short periods without additional binder. A second application of the emulsion is then sprayed at a rate of 2 – 3 L/m^2. This produces a surfacing that is strong and climate-resilient. The binder plays a lesser role in binding the aggregate because they are held together mostly by the wedging effect. The binder is therefore less stressed and prevents cracking and deformation from occurring.

Some key solutions are illustrated in Figure 12.7 and Figure 12.8.

Figure 12.7 Typical Section of Urban Roads With Sub-Surface Drainage and Amalgamated Surfacing for Climate Resilience

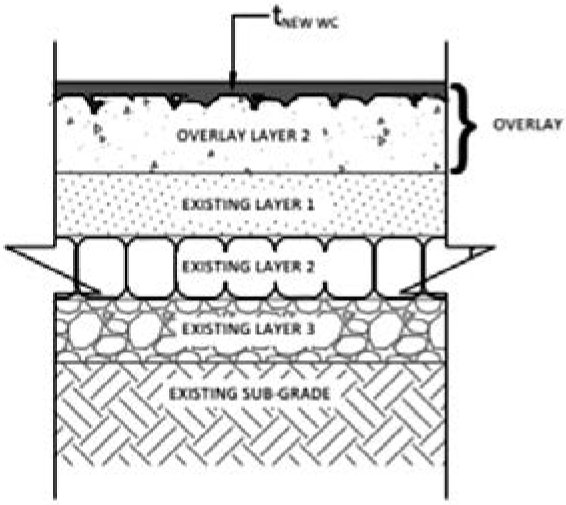
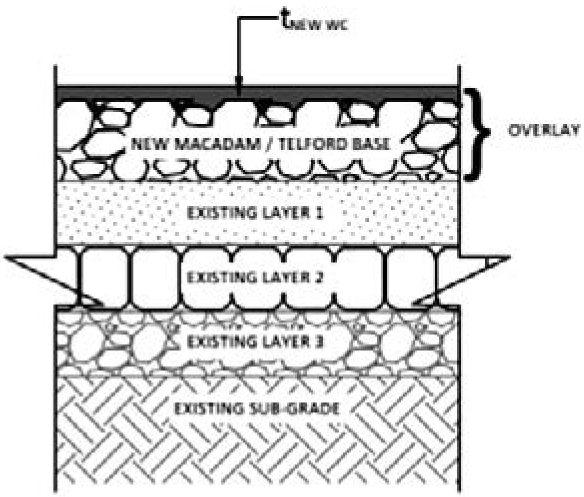


Figure 12.8 Typical Macadam Base With Amalgamated Surfacing for Climate Resilience



12.5.8 Catalogue CMM1 – Reducing Carbon Footprint of Road Building Materials

12.5.8.1 Challenge

Road-making materials contribute to the carbon footprint in several ways, and should be considered by the design engineer.

Acquisition - Acquisition of materials is a major operation, and the contribution can be evaluated depending on the category of the materials.

1. Natural materials – this includes the key stages:
    a. The extraction of materials i.e., gravels, soils, sands, stones and quarries – involves equipment, which uses fossil fuels. Heavy machinery is a major contributor. For the natural soils and gravels, removal of overburden, excavation, stockpiling and transportation are the main activities contributing to emissions.
    b. For crushed rock – it involves blasting, crushing and stockpiling.

2. Bituminous materials:
   a. Oil mining – oils that are preserved in the ground are brought to the surface. This carbon footprint is significant. Petroleum gases are released into the atmosphere.
   b. Fractional distillation releases fumes and the heating required produces GHGs.
   c. The manufacture of binders by the manufacturers increases an already high carbon footprint.
3. Additives like cement and lime:
   a. The extraction is equipment intensive and heavy machinery is used.
   b. The manufacturing is heat-intensive and has a significantly high carbon footprint.

Reducing the carbon footprint of materials used for road projects is a serious engineering challenge because most of them are produced and transported using means that have a high carbon footprint.

12.5.8.2 Solution

Some of the activities and processes can be decarbonised through:

1. Extracting fewer materials by modifying design approaches to minimise material quantities needed in overlays and pavement strengthening.
2. Recycling of materials.
3. Designing lean pavements.
4. Using materials with a low carbon footprint.
5. Reducing the use of asphalt or thick asphalt.

The following details cover means of mitigating climate change for sustainability.

12.5.9 Catalogue CMM2 – Transportation

12.5.9.1 Challenge

Transportation is a high carbon footprint activity. Haulage trucks are mostly diesel-powered and fuel consumption is very high when trucks are laden. Additionally, the trucks are emptied on return trips to the source of the materials but still emitting GHGs. The landed carbon footprint of the materials is generally high.

12.5.9.2 Mitigation Solution

1. Use locally available materials – Use the principle of 'rehabilitation design that promotes the use of locally available materials and not materials for the design'. This means that the design engineer should modify designs for maximum use of locally available materials so that the transportation of materials is not over long-haul distances, significantly reducing the carbon footprint of the materials.
2. Bulk transportation – the transportation of materials in bulk reduces the carbon footprint per cubic metre of the materials.
3. Condition of the route – poor condition of the roads to the sources of the materials means the truck will tend to use heavy gears and consume more fuel, so maintaining the access or road to the source of the materials will reduce the carbon footprint.
4. Speed – moderate speed is good, but very high or very low speed increases the carbon footprint.
5. Condition of vehicles – usually vehicles work in harsh conditions and would normally be in a bad state of repair. Maintenance of the vehicle fleet will help to minimise the carbon footprint.
6. Recycling – in rehabilitation, recycling helps to reduce the demand for transportation of materials and hence the carbon footprint.

12.5.10 Catalogue CMM3 – Construction Process

12.5.10.1 Challenge

The construction process also contributes significantly to the carbon footprint. The contribution depends on the type of materials being used. Some materials require heating and processing for prolonged periods. The following are key contributors:

1. Surface dressing and asphalt mixing and laying require heating thus contributing to GHGs.
2. Bituminous materials when heated produce hydrocarbon fumes which are key pollutants and contribute significant amounts of GHGs.
3. Processing of cement concrete and other pavement materials involves the use of heavy plant and is a major GHG contributor.

12.5.10.2 Mitigation Solution

There are several ways in which the construction process can be modified to reduce its carbon footprint.

1. Using cold and warm mix asphalt reduces the heating required, resulting in a low carbon footprint.
2. Using emulsion for emulsion-based Otta/graded seals, sand seals, slurry seals and grit seals requires no heating and is largely green construction.
3. Amalgamated penetration macadam surfacing with 20 mm to 40mm aggregate for the first layer and a second layer of nominal maximum size of 13 mm graded aggregate plus application of bitumen emulsion stable 60 (K2) forms a strong penetration macadam surfacing. Only a half to a 3rd of the standard binder application of 4 L/m^2 -6L/m^2 is required. This significantly reduces the carbon footprint of penetration macadam and the overall construction costs.
4. Method of construction - chose greener options for the method of construction:
   a. Promote labour-based approaches for the construction of roads and other infrastructure for most of the low-volume roads.
   b. Promote the use of intermediate technology in infrastructure development – involves using a balanced mix of labour and machines with a higher preference for labour.
5. Use of greener fuel for equipment – consider the use of greener fuels for the plant such as
   a. Plant-based blends of diesel and petrol which have lower carbon footprint.
   b. E-plant and equipment – using electrically powered plant and equipment will greatly reduce the carbon footprint. This will, however, be difficult to achieve where the power supply is unreliable or in remote areas.
   c. Hydrogen fuel – is a new technology that is under development. It is clean unless the production of hydrogen emits GHGs.
   d. Use of efficient plant and equipment – using equipment and plant that are efficient and in good state of repair lowers their carbon footprint.
6. Materials for ease of construction – there are additives that can be applied to materials such as concrete to improve workability so that less effort is required thus reducing the carbon footprint of the construction process.

12.6 Planning for Climate Resilience Intervention

Several steps should be followed through the planning process.

1. Determine the risk level for the road and project area.
   a. Collect flood risk maps and locate the project area. Also, collect rainfall maps showing the distributions of average annual and maximum daily rainfall.
   b. Check the Intensity, Duration and Frequency (IDF) curves for the project area and that of the catchment bearing in mind that pavements can be flooded from runoff and rivers carrying stormwater from very far away and even neighbouring countries. Consider that rivers can burst their banks and flood the pavements.
   c. Refer to the Hydrology and Drainage Design Vol. 2 of the RDM for further information on potential extreme flood frequency and impacts.
   d. Collect temperature maps and consider the maximum daily and average annual maximum daily temperatures. Determine the period or duration in days when these temperatures occur.
   e. Determine the annual exceedance probability i.e., the probability of exceeding the average maximum values of precipitation and temperatures. These can be considered extreme weather conditions.
   f. From the hydrology and drainage design obtain information on the maximum probable flood (MPF).
   g. Collect climate change maps showing predictions of changes in precipitation and rainfall intensity for pavements; 30 years for LVRs to 50 years for HVRs.
   h. Collect maintenance records and performance history of the existing pavements.
   i. Collect information on extreme weather events from the communities and road authorities. This information should include,
      i. Types of flooding – sheet flooding or channelised flooding.
      ii. The impacts of any previous extreme weather events, like areas that got overtopped.
      iii. Impact of climate change on the infrastructure.
      iv. Durability of pavements and surfacing.
      v. History of pavement failures, poor performance and mode of failures.
   j. Produce an inventory of the road in general and pavements specifically to determine the types and structures of existing pavements and pavement materials used.
   k. Assess the current condition of the road.
2. Using the information obtained, produce a strip map of the road and road pavements demarcating areas of high risk and vulnerability following guidance in Section 10.3.2. The information should cover the nature and severity of the climate risk and vulnerability levels
3. Determine the probability of exceedance of the average maximums of rainfall intensity, annual rainfall, and temperature. Use any available relationships or develop trends from the data and extrapolate using values of climate change predictions provided on the maps. If the climate change maps are not available calculate the most probable flood or obtain the values from the hydrologist.
4. Using the guidance below assign the most appropriate climate resilience solution options for each section of high vulnerability.

1
2
3
4
5
6
7
8
9
10
11
12
Considerations for Climate Resilience & Mitigation

5. Calculate the additional costs associated with the climate resilience interventions or retrofitting.
6. Determine the value of the road asset and the pavement in particular.
7. Estimate the impacts of climate change on pavement performance using the historical performance data and software such as HDM4, or RED Model or the Life Cycle Costing (LCC) Model to predict performance and maintenance requirements.
8. Using the predicted recurrence of extreme weather events estimate the damage that can be anticipated as a result, i.e., without consideration of the resilience interventions.
9. Estimate the life cycle costs with and without climate resilience interventions. The difference is the cost of a do-nothing scenario, and the savings are benefits which accrue when a climate resilience design approach is incorporated in the designs or retrofitting climate resilience for vulnerable sections is considered.
10. Determine the Net Present Value of all costs calculated, Equation 12.2.

$$NPV = \sum \frac{P}{(1+i)^t} - C$$

Equation 12.2

Where,

P = Net period cash flow.

i = Discount rate or rate of return.

t = Number of time periods.

C = Initial investment.

11. Carry out a benefit-cost analysis to determine the viability and justification of the interventions.
12. Include climate resilience in project plans with the associated funding requirements.

12.7 Design and Implementation of Climate Resilience Measures

This section covered some of the key engineering solutions for consideration in rehabilitation design for climate resilience. Key changes to the design standards are given below.

12.7.1.1 Pavements Prone to Flooding

For pavements that are prone to flooding and inundation for long periods consider the following:

1. Significant weakening of the pavement layers and subgrade – refer to CPM1 and CPM2.
2. Delamination of surfacing/wearing courses – wearing courses delaminate when submerged and trafficked soon after the flood. To prevent this, design amalgamated surfacing:
   a. For double surface dressing – use less than half the aggregate size of the first layer for the second layer. This amalgamates the two layers into one avoiding a plane interface between the two.
   b. For thin asphalt – apply the asphalt on a base with protruding stone. Some of the asphalt is pressed into the interspaces. This amalgamates the AC with the base to function as one layer.
3. Scour and severe damage.
   a. For fine subbases (e.g., sand bases) armour the side slopes to prevent scouring.
   b. For side slopes that are prone to erosion – construct side beams to confine the pavement and prevent scouring from storm water in the side drains.
   c. In cases of overtopping – provide edge keys and reinforced cladding on the side slopes. The side slopes should be gentle typically 1:4 or 1:5 with an apron down slope.

4. Armouring of natural bases for LVSRs and HVSRs.
   a. Design for armouring of bases for LVSRs – armoured bases are normal granular fine-grained bases with a single layer of armouring stone of nominal size 40 mm – 50 mm hammered into the base (see Figure 12.7 and Figure 12.8). During construction, the surface of the base is scarified slightly to about 20 mm depth. The layer of crushed stone is applied on top and hammered into the base using very heavy compactors vibrating at high amplitude. Hammering/compaction is carried out till refusal. At this stage, no heavy load can push the stone any further into the base and this is the armouring effect, also called micro-piling, to form an armoured base.
   b. Design armoured bases for HVSRs – armouring of bases on high-volume roads is similar to that of LVSRs. The difference is that for HVSRs the armouring layer is thicker. The thickness can be 100 – 150 mm. The base is scarified to a depth of 50 mm. Larger stones of nominal size of 50 – 70 mm are applied and hammered to refusal. A second layer of 20 – 40 mm aggregate is applied on top and vibrated into the interspaces of the first layer with rollers operating at low amplitude.
   c. Design for macadam or Telford bases – bases with characteristically large stone aggregates, up to 150 mm. They are resistant to washaways and damage during flooding.

12.7.1.2 Design for Extreme Temperature

Design considerations of extreme temperatures are for temperature-sensitive materials and pavements. This includes most AC and binder courses such as AC-I and AC-II, DBM and thin and micro-surfacing. Temperature is an inherent parameter of design including selection of the binders for application, types of surfacing, etc.

The design process for bituminous layers and surfacing is well known and there are only a few challenges in this regard, because of the experience gained over long periods of use. However, there is a new challenge of significant temperature increases due to climate change. The effects are less severe for the gradual increase in temperatures than those caused by extreme weather events. The principles of consideration of temperature include:

1. Maximum temperatures.
2. Duration or period when the maximum temperatures occur e.g., days in a year.
3. Conditions of extreme weather events.
4. Maximum probable temperature.
5. Annual exceedance probability for average maximum temperature.

There are different scenarios a section could be evaluated for:

1. Softening of binders in surface treatments at high temperature – when binders in the surfacing soften bleeding and/or dislodgement of aggregate occurs.

   Design solution:
   a. Application of grit of nominal maximum size of 9mm.
   b. Raked in stone rolled into the surfacing to increase the skid resistance.
   c. Reseal with a harder binder 80/100 pen or 60/70 pen, 50/60 pen. Binders harder than 50/60 pen should not be considered for surfacing.
2. Softening of binders in asphalt layers

   Design for:
   a. DBM for binder course made of hard binder which 40/50 pen or 30/40 pen.
   b. Open-graded AC for high rut resistance with surface dressing on top. Include a maintenance plan for fog sprays and reseals within 4 – 7 years respectively.

3. Rapidly cracking wearing course

   Design solution:

   a. Open-graded AC wearing course for rut resistance with thin surfacings on top to prevent further deterioration from oxidation.
   b. Thin AC of medium grading and well compacted is amalgamated with stoney base or macadam or placed on cement stabilised base.
   c. Thin surfacing or thin AC wearing course on macadam or Telford or coarse cement stabilised bases exhibiting low deflections (< 200 microns) making them more resilient.
   d. Use polymer-modified binders in the thick surfacing or AC wearing course or both the wearing course and binder course. Polymer-modified binders are heat resistant.
   e. Use fibre mastic asphalt (FMA) – the fibre helps to minimise cracking of the asphalt. This minimises the deterioration of the asphalt.

13 Design Considerations for Urban Roads

13.1 General

This section covers maintenance and rehabilitation design considerations for urban roads. The design process is similar to the processes detailed in previous Chapters, relating to pavement evaluation, design for maintenance and rehabilitation including methods of overlay design for flexible and rigid pavements. This section provides guidance on design considerations particular to urban roads.

13.2 Urban Roads Classification for Rehabilitation Design

This section covers considerations for road classes in determining maintenance requirements and rehabilitation of urban and peri-urban roads. The requirements for condition surveys and design standards differ significantly for the different classes of roads and this needs to be considered for sustainability and economic provision of serviceable urban roads.

Urban roads are provided in 4 main categories given in Table 13.1. The approach used for rehabilitation design differs depending on the class of urban road.

Table 13.1 Urban Roads Classification for Pavement Maintenance and Rehabilitation Design

Category	Road Class	Types of Roads	Function
1	**Principal Arterial**	Freeway, motorway, expressway, highway, national road, trunk road, primary road,	This is a vehicle-only route and for high-order mobility. Carries heavy traffic and is usually multi-lane.
2	**Collector street**	Collector, major collector, minor collector, local distributor, street, high street, busway, connector, district road, rural road, rural access.	For inter-municipal, industrial local distribution, integration and collection. Mixed pedestrians and traffic.
3	**Local street**	Street, local street, access street, local residential street, residential road, rural tertiary road, rural access road.	Individual access via access collectors, loops, cul-de-sacs, etc.
4	**Walkway**	Pedestrian priority and pedestrian-only street, parking, pedestrian walkway, and path.	Non-motorised access

13.3 Key Technical Differences between Urban and Rural Road Pavements

Urban roads differ from their rural counterparts in several aspects, Table 13.2.

Table 13.2 Consideration of Differences Between Urban and Rural Roads

Design Aspects	Urban	Rural
Traffic mix	Dependent on the functionality given above. There is a need to cater for the separation of vehicular traffic from pedestrians and non-motorised traffic	Traffic is less diverse.
Services	The pavements are interlaced with service lines such as water reticulation, sewer, manholes, communication lines, power cables etc.	It is usually rare to encounter service lines in or under the pavement.
Storm drainage	Storm drainage systems within and outside pavement structures are prevalent occurrences and should be considered in rehabilitation design.	Drainage systems are usually surface and subsurface
Levels	Final road levels should not change significantly because of the need to maintain the elevations of kerbing, manholes, and height clearance for multilevel road systems, access to properties, etc. This poses a challenge for overlay design, so inlays are preferred.	Overlays tend to increase the height of the final road levels which is generally a positive thing for rural roads or highways in rural areas
Roadworks	Urban roads have high peak flow and closures tend to be very disruptive. Construction activities must be completed, and the road should be opened to traffic quickly. There is no option for detours.	It is possible to construct detours hence the disruption to traffic is minimal

13.4 Design Considerations of Urban Road Pavement Structures and Services

This section provides guidance on key considerations of the pavement structures for major and minor roads and the service lines that run under and on the periphery of pavements in urban areas. These aspects must be considered during pavement condition investigations, and maintenance and rehabilitation design. The pavement structure and configurations are influenced by the class of roads given in Table 13.1. An illustration of the structure and services is given in Figure 13.1 for major roads and Figure 13.2 for minor peri-urban roads.

Figure 13.1 Illustration of Urban Pavement Structures and Service Lines

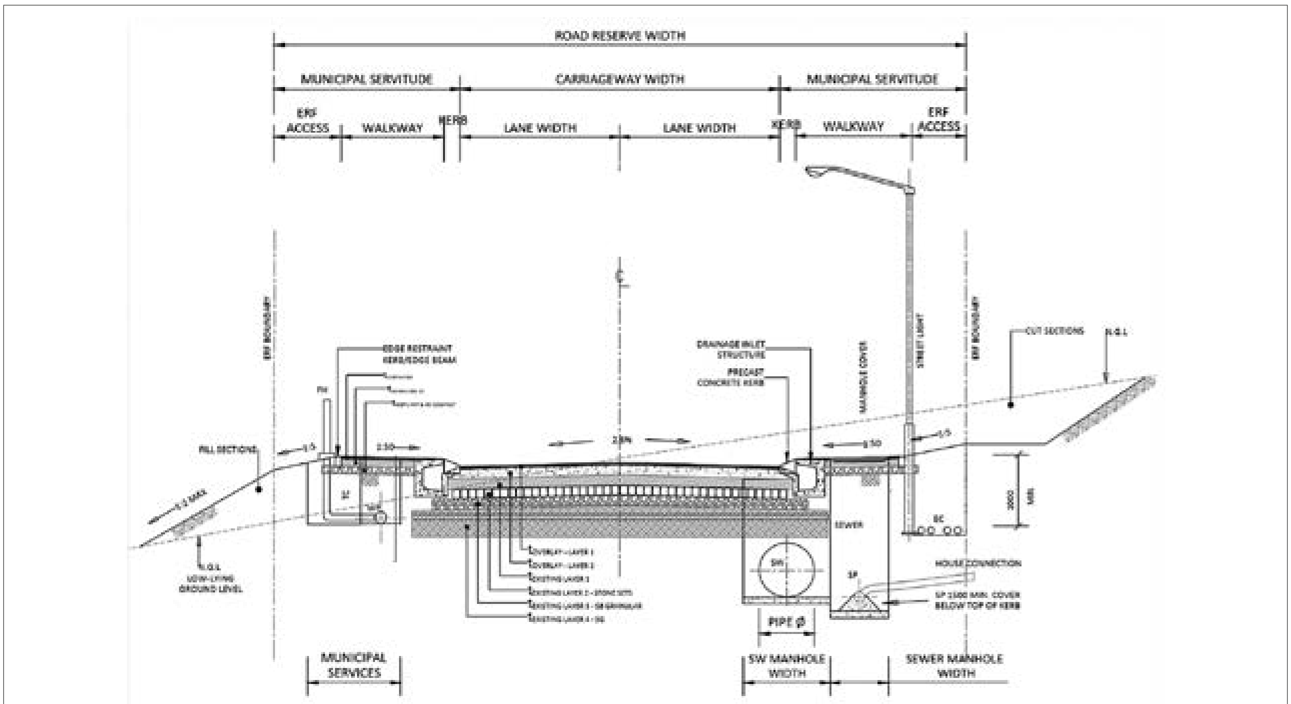
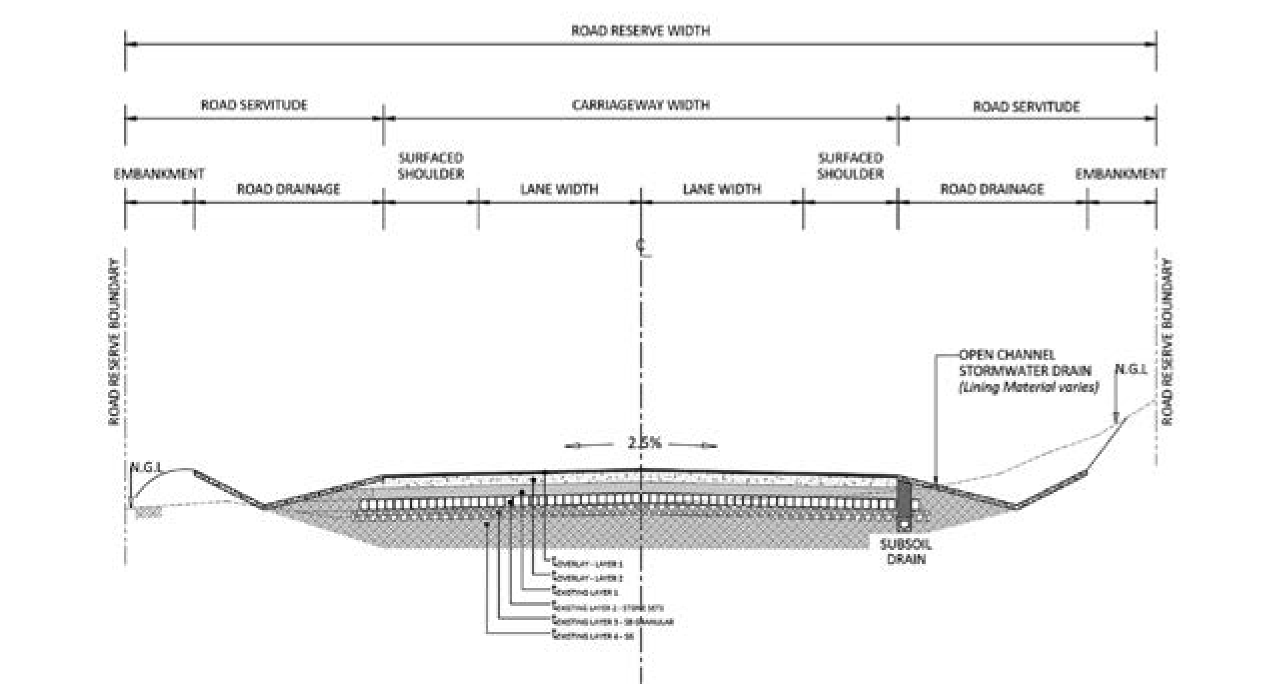


Figure 13.2 Illustrations of Peri-urban Pavement Structures and Services



Urban road pavement structures need special considerations which are given in Table 13.3:

Table 13.3 Key Pavement Rehabilitation Design Considerations for Urban Roads

Item No.	Design Aspects	Urban
1	**Types of pavements**	Consideration is made of the road pavement and the walkway. These are designed to different standards and require different materials for their construction. Their rehabilitation is generally influenced by the need to maintain levels and appropriateness of the materials for future recycling and longevity.
2	**Pavement layers**	Some of the pavement layers may have been constructed at different times through the evolutional development of the road. Their condition needs to be determined earlier on in the investigations.
3	**Surfacing or wearing course types**	This is critical on city roads motorised and non-motorised traffic, and pedestrians. Rough surfacing on walkways forces pedestrians onto the carriageway hence smooth surfacing is preferred.
4	**Traffic distribution**	It is highly dependent on the number of lanes and lane widths and traffic counts need to be carried out per lane and direction, see Chapter 5 for the determination of the lane distribution factors Refer to RDM 1.2 and RDM 3.3.
5	**Service lines**	Service lines under and in the periphery of the pavements should be investigated to determine their depth, position, orientation, nature and fragility. It should be determined how they will be impacted by pavement rehabilitation or how their existence will impact rehabilitation. Bus rapid transit routes – axle loading is severe, and a different standard axle load should be considered (130 kN).
6	**Industrial roads**	The design should consider traffic carrying abnormal indivisible loads like plant and machinery and heavy goods like ore, steel, cement, etc
7	**Drainage**	Drainage is usually a problem in urban areas. Overlays raise levels of pavement surfaces, affecting storm drainage including manholes, graters and other inlets, which may need to be lifted. Poor drainage may affect the performance of the road pavements, significantly. Rehabilitation should consider the improvement of drainage. Pavement overlays or inlays should be designed for the use of low moisture-sensitive materials, see Section 12.5.6 on how to determine moisture sensitivity of materials.

1 2 3 4 5 6 7 8 9 10 11 12 13 Design Considerations for Urban Roads

Item No.	Design Aspects	Urban
8	**Speed**	Speed on urban roads is generally low. If a road carries heavy slow-moving traffic, overlays or inlays should be designed for severe in-service conditions. Congestion exacerbates the severity because of the long duration, vehicles are stationary or slow-moving during congestion. The design should consider overlays or inlays that are made of rut-resistant materials.
9	**Long life pavements**	Consideration for maintaining existing final road levels means inlays are preferred. Traffic growth should be considered and pavement strength should be improved with the increase in cumulative traffic loading. This means that rehabilitation design should consider long-life pavements for the different classes of roads. For high-volume roads the design for 80 MCESA is considered adequate strength for long-life pavements, see Section 9.6.4. Categories of long-life pavement design are given in Table 9.7.
10	**Materials**	Materials used for the rehabilitation of urban road pavements should generally be recyclable. GCS, natural gravels, and discrete elements such as stone sets, cobblestones, paving blocks, and asphalt can be recycled. If the rehabilitation design considers recycling, cement stabilisation should be avoided where possible.
11	**Pavement evaluation**	*1.* The level of pavement evaluation should depend on road class: *a.* For categories 1 to 2, full surface and structural condition surveys should be carried out. *b.* For categories 3 and 4, surface condition surveys should suffice. The pavements will most likely fail due to environmental factors than loading.
12	**Rehabilitation design**	*1.* Whether AC is present or not, rehabilitation design should involve an AC overlay thick enough for future milling and replacing (inlays). *2.* Where other recyclable materials exist in the base or wearing course, recycling shall be preferred. *3.* Where strengthening is required to cater for high future cumulative traffic loading: *a.* The layer(s) i.e., the base course and wearing course can be strengthened through modification or stabilisation. *b.* Alternatively, alter the pavement structure to increase the base thickness and reduce the thickness of the subbase. This approach, increases the strength of the pavement without changing the existing final road levels. For categories 1-2, use the Mechanistic Design Software to carry out design iterations to determine the most appropriate adjustment of materials and layer thicknesses.

13.5 Design Procedure

This section provides a summary of the design procedure for the rehabilitation of urban roads.

13.5.1 Pavement Condition Surveys

The procedure for the pavement condition surveys should be conducted as provided in RDM 5.1. Key considerations for urban roads include:

1. Carrying out visual condition surveys:
    a. For categories 1-2 carry out the condition surveys described in RDM 5.1.
    b. For categories 3-4 distinguish between damage and deterioration. This is because the main causes of deterioration are not load-related, but mainly environmental. This may include damage caused by straying heavy motor vehicles or very weak or expansive subgrade or deteriorated surfacing due to oxidation and loss of volatiles. AC can crack from environmental causes only.

   c. Categories 3 and 4 can be surfaced using discrete elements like cobblestone, stone sets, burnt clay brick, concrete block paving, etc. Condition surveys involve checking for breakages, dislodgements, siltation, deformation, clogging, etc. Most defects are localised.

2. Carry out GPR surveys – these are critical for detecting underground services. Correct interpretation of the GPR imagery and data is paramount, see RDM 5.1. Some of the services could be very high value and difficult and/or expensive to relocate when necessary. GPR surveys can be used to determine water tables especially when they rise to SG and pavement levels, which may cause pavement deterioration or failures.

3. Carry out structural condition surveys:

   a. For categories 1 and 2, surveys should be carried out as specified in RDM 5.1.

   b. For categories 3 and 4, structural condition surveys are generally unnecessary but may be carried out upon request by the responsible authority. Detailed surface condition surveys will suffice.

   c. For discrete elements, check that the elements are sitting on a sand blanket and that a subbase is of G20 quality and 100 – 150 mm thick. The sand blanket should not be clogged with clay or silt. There should not be deferential displacement of the elements. No overlaying is necessary unless thin asphalt is required to smoothen the ride quality.

13.5.2 Rehabilitation Design Procedure

Rehabilitation design should be carried out using the methods specified in Chapter 9 and the key considerations given in Section 13.4.

13.5.2.1 Rehabilitation Design of Principal Roads (Expressways) and Arterials

Below are the key design approaches that should be considered.

1. Consider inlays if pavement design capacity is for 80 MCESA unless specified otherwise. This is based on the principle of long-life pavements that the pavement for 80 MCESA is thick enough to protect the subgrade against any further passages of the 80 kN ESA. Generally, above the 80 MCESA, no further increase in the thickness of the pavement is required.

2. For concrete viaducts, design for AC1 fatigue life to determine the thickness of the wearing course over the concrete decks, see Section 9.6.

3. For roads at level, design for overlay following methods given in Chapter 9.

4. For minor arterial roads, design for rehabilitation of raised walkways with fill material for SG, subbase of 20% CBR (min G20) and surfaced with ACII, or double surface dressing 7/13 or discrete elements. If it already exists and is in poor condition, design for recycling of the base and seal.

13.5.2.2 Rehabilitation Design for Connector Roads

Connector roads include industrial roads which may be heavily trafficked. There is also the mix of pedestrians and traffic, which for safety, should be separated. For the rehabilitation of the carriageway, methods given in Chapter 9 should be used, and for walkways follow the guidance in Section 13.5.2.1 Item (4).

Connector roads also include bus lanes, which should be designed in the conventional way (Chapter 9).

A bus rapid transport route is also considered as a connector road. Heavier loads are assumed for the design. A 130 kN standard wheel load is assumed hence the design for the pavement should be considered differently from the normal design of 80 kN equivalent standard axle.

1 2 3 4 5 6 7 8 9 10 11 12 13 Design Considerations for Urban Roads

13.5.2.3 Rehabilitation Design for Local Streets and Walkways

Usually, these roads are lightly trafficked, and the rehabilitation design should be carried out differently. The design engineer should focus more on environmental and drainage issues as well as the condition of the surfacing. The design process should include aspects given in Table 13.4.

Table 13.4 Design Consideration for Urban Local Streets and Walkways

Item No.	Design Aspects	Urban
1	**Condition Surveys**	*a.* Check drainage for high/patched water tables, signs or history of flooding, condition and capacity of the drainage systems, subsurface drainage, possible water percolation into the pavement, etc. *b.* Investigate the subgrade – for the potential for high expansivity and other problematic characteristics like dispersion, salinity, collapsibility, etc. Also check the densities. *c.* Investigated pavement layers – for densities, degradation (e.g., of basaltic soils and aggregate), deformation, erosion, etc. *d.* Check the age and condition of the surfacing, i.e., cracking, spalling, potholes and patching, and other surface defects. Such minor roads fail due to the deterioration of the surfacing through ageing. This should be the main focus for the design engineer. Ingress of water through breached surfacing is the main cause of potholing and deformation.
2	**Structural condition surveys**	*a.* They are generally unnecessary, but should they be required only DCP and/or deflection tests should be carried out. *b.* Additionally, Benkelman Beam or LWD can be considered for deflection tests. *c.* The design engineer to decide the structural investigation method to use
3	**Rehabilitation design**	*a.* If cracking is just starting – apply fog sprays. *b.* If cracking has advanced – seal the cracks and apply a thin reseal or thin surfacing. *c.* If the surfacing and base have disintegrated – remove the surfacing, recycle the base material (rework the base materials and recompact to level). Apply new surfacing.

14 Considerations for Unpaved Roads

14.1 General

Unpaved roads constitute the greatest part of road networks in most road authorities in LICs and MICs and Kenya in particular. They service the larger part of communities in rural areas. They are the major network supporting economic production in agriculture, mining, fishing, tourism, etc.

Generally, unpaved roads constitute more than 80 % of the network. They have the greatest demand for natural materials including gravel for road construction and rehabilitation which is generally scarce.

There are several key considerations for their sustainable provision:

1. The capital costs of new construction – involving the opening up of new access
2. The maintenance and rehabilitation of rural unpaved roads.
3. Upgrading of unpaved roads to sealed road standards.

This section covers the maintenance and rehabilitation of unpaved roads and the economic decision to upgrade to sealed road standards when it is no longer sustainable to maintain the road as gravel or earth.

14.2 Structure of Unpaved Roads

The structure of unpaved road pavement is simple and constitutes of components given in Table 14.1.

Table 14.1 The General Structure of Unpaved Roads

Layer No.	Layer	Description
1	**Wearing Course**	Granular material with a minimum CBR of 20 %, PI between 5 and 20 but mostly defined using the plasticity product (PP) and plasticity modulus (PM). The material should be medium-graded, i.e., not too coarse or fine. The grading modulus should be between 1.0 and 1.9, but can be as much as 2.5, where medium-graded materials are scarce. For performance-based standards, see Figure 14.1.
2	**Subgrade/ Foundation**	Fine to granular soils with a minimum CBR of 11 %. Lower CBRs like 5 % can be acceptable for lightly trafficked roads
3	**Capping**	Capping is necessary in situations where the in-situ SG is very weak CBR < 5 % and highly plastic (PI > 20) and PP > 600.

14.3 Pavement Evaluation

Maintenance and rehabilitation of the pavement is important for its sustenance and economic viability. Consideration of deterioration of unpaved roads involves:

1. Rate of gravel loss.
2. Rate of roughness progression.

Pavement evaluation involves the following:

1. Checking defects:
   a. Functional defects – slipperiness, corrugations, potholes, roughness, etc.
   b. Structural defects – deformation, gravel loss, erosion, wearing course thickness, etc.
2. Check environment:
   a. Check drainage – scouring, water logging, the capacity of the drainage system, and overtopping of the road.
   b. Traffic – composition and general traffic speed.

14.4 Maintenance and Rehabilitation Design

14.4.1 Failure Criteria for Unpaved Roads

It is important to define failure criteria for unpaved roads Table 14.2:

Table 14.2 Failure Criteria for Unpaved Roads

Item No.	Criteria	Specification	Considerations
1	**Roughness**	For high volume roads = 6 max.	Threshold for maintenance grade intervention
		For low volume roads = 8 max.	Threshold for maintenance grade intervention
2	**Thickness of gravel**	50 mm minimum	Threshold for regravelling intervention
3	**Grading cycles**	More than 2 times per year for traffic < 100 vpd	Threshold for change of material quality
4	**Grading cycles**	More than 2 times per year for traffic ≥ 100 vpd	Threshold for upgrading to sealed road standards
5	**Regravelling cycles**	More than once in 3 years for traffic < 100 vpd	Threshold for change of material quality
6	**Regravelling cycles**	More than once in 3 years for traffic ≥ 100 vpd	Threshold for upgrading to sealed road standards

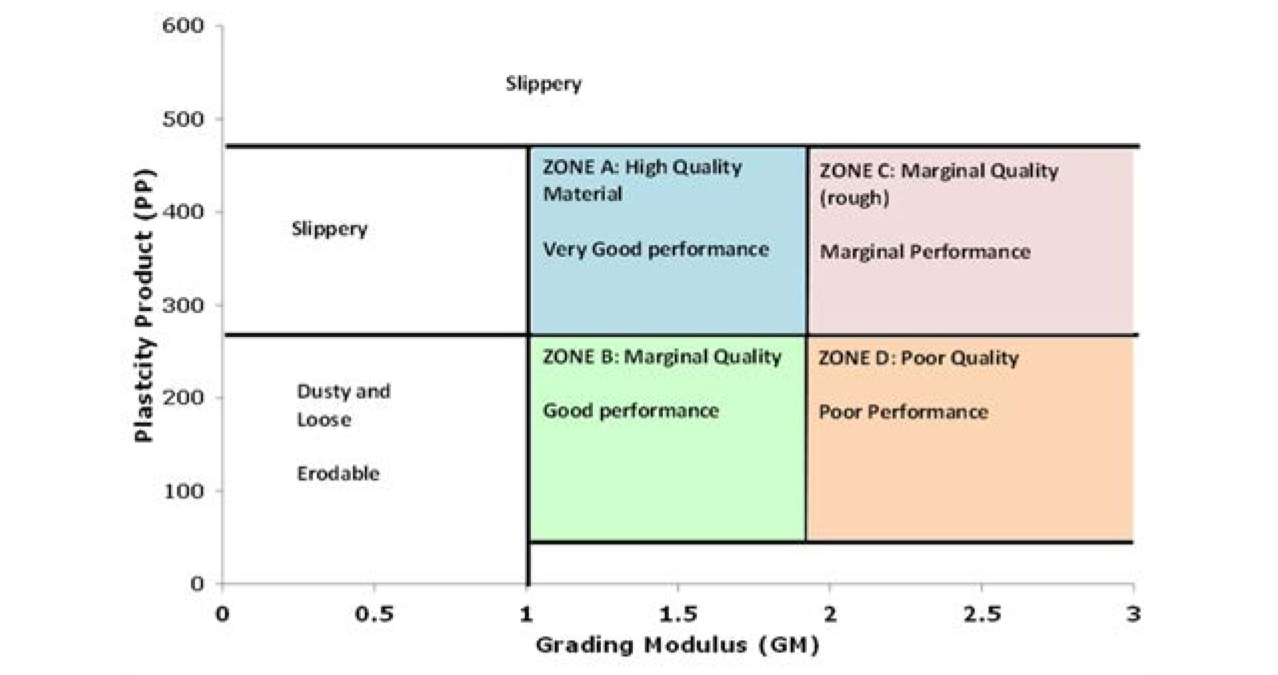
14.4.2 Regravelling (Overlay) Design using Performance Based Specifications

Regravelling is a form of overlay. The design of the overlay is based on the material performance regarding the rates of gravel loss and roughness progression.

14.4.2.1 Design Procedure

1. Use the chart in Figure 14.1 to select the best material available for the regravelling (overlay).

Figure 14.1 Performance-based Specifications for Gravel Wearing Course (K. Mukura et al)



Where,

a. PI = the Plasticity Index of the material passing the 0.425 mm sieve.

b. Plasticity Product (PP) = PI x $P_{0.075}$. The preferred range is 280-480.

c. Grading Modulus (GM), given in Equation 14.1.

$$GM = 3 - \frac{P_{2.36} + P_{0.425} + P_{0.075}}{100}$$

Equation 14.1

Where,

$P_{2.36}$ = Percentage passing the 2.36 mm sieve.

$P_{0.425}$ = Percentage passing the 0.425 mm sieve.

$P_{0.075}$ = Percentage passing the 0.075 mm sieve.

The preferred range is 1.0 - 1.9.

The particle size distribution test for the material must be done using the wet sieving method.

2. Select the failure criteria for minimum gravel thickness (e.g., 50 mm) and max. IRI for unpaved (e.g. 6 m/km). These are used as thresholds for triggering regravelling and maintenance grading respectively.

3. Calculate the traffic volume in vehicles per day (vpd). The rate of gravel loss is influenced predominantly by the volume and speed of the traffic rather than the axle loads, contrary to sealed roads. Light vehicles travelling at high speeds cause more gravel loss than heavy slow-moving vehicles.

4. It is important to consider the following parameters to develop life-cycle costs:

   a. Capital cost of construction per km

   b. Cost of routine maintenance per km

   c. Unit cost of maintenance grading per km

   d. Unit cost of regravelling per km

   e. Calculate life cycle costs (LCC) using Equation 14.2.

1
2
3
4
5
6
7
8
9
10
11
12
13
14
Considerations for Unpaved Roads

$$LCC = capital\ costs + maintenance\ costs$$

Equation 14.2

5. Maintenance costs include grading, regravelling and routine activities like pothole filling.
6. Use the life-cycle cost model LCCM to determine strategic future interventions.
   a. Use the selected values for the failure criteria e.g. 50 mm minimum thickness after wearing away of the gravel and 6 m/km maximum IRI.
   b. Determine the rates of gravel loss from historical data.
   c. Determine roughness progression also from historical data.
   d. If historical data are available consider the average values of rates of gravel loss and roughness progression.
   e. Using the cumulative traffic volume over the analysis period determines regravelling cycles and maintenance grading cycles in years, see example Figure 14.2 and Figure 14.3. And determine life cycle costs, Figure 14.4.

Figure 14.2 Example of Maintenance Grading Cycles

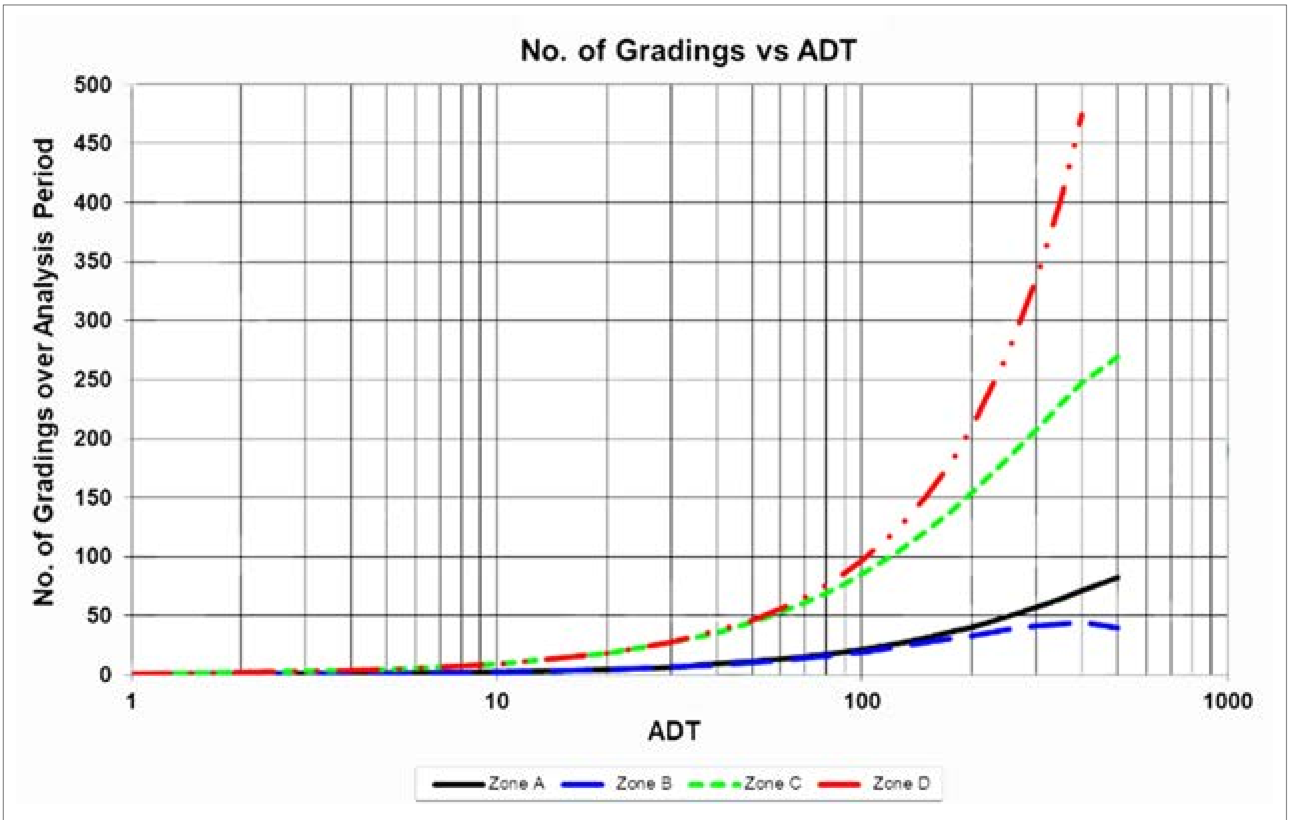
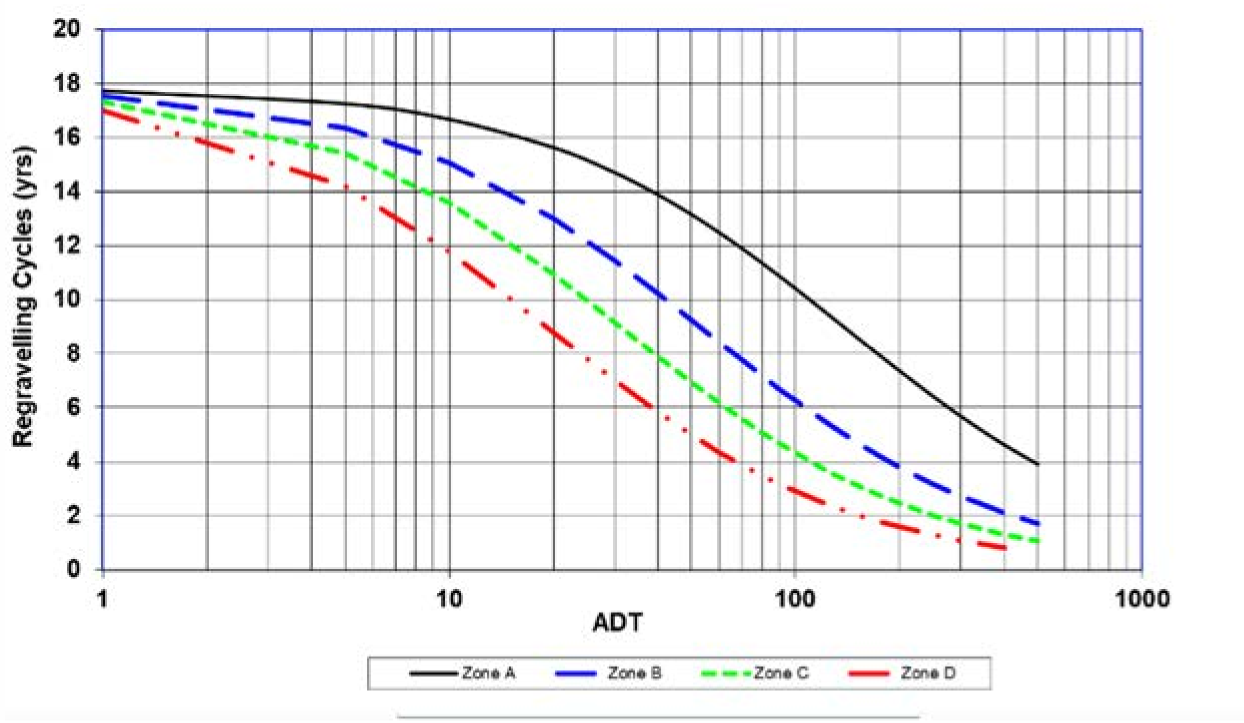
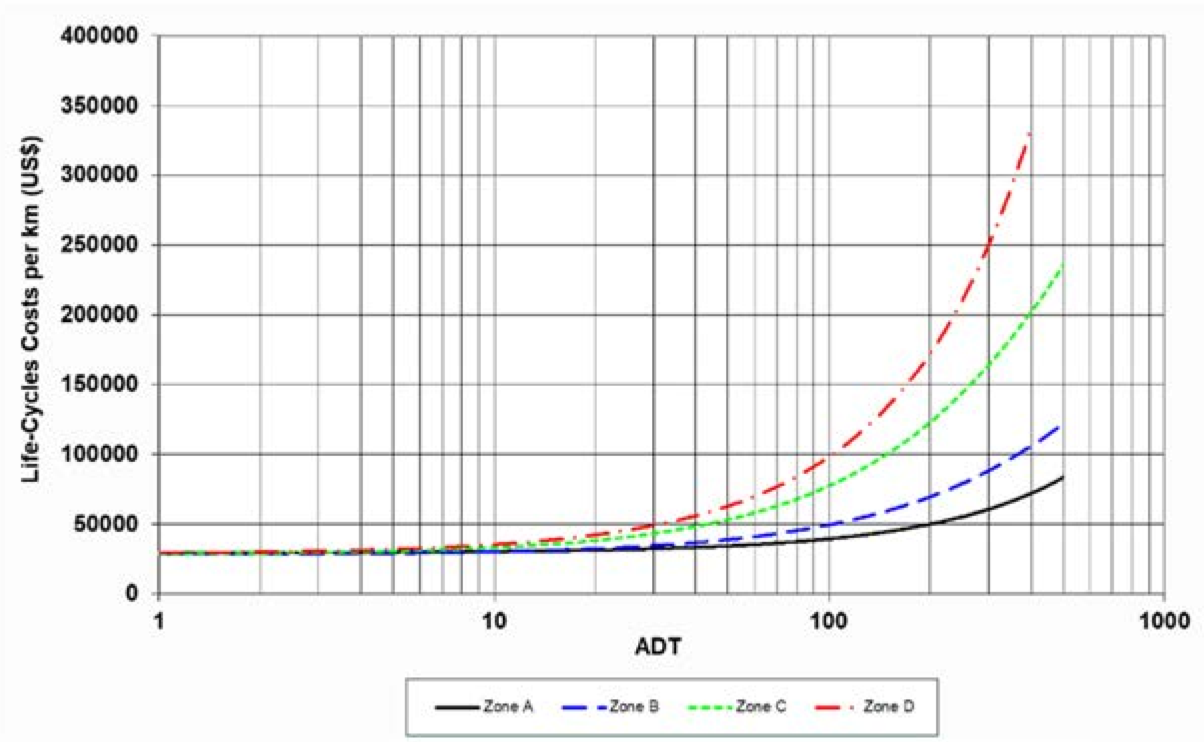


Figure 14.3 Example of Regravelling Cycles



Read off the life-cycle costs over the analysis period. Life cycle costs are calculated and can be plotted against ADT, Figure 14.4.

Figure 14.4 Example of Life Cycle Costs



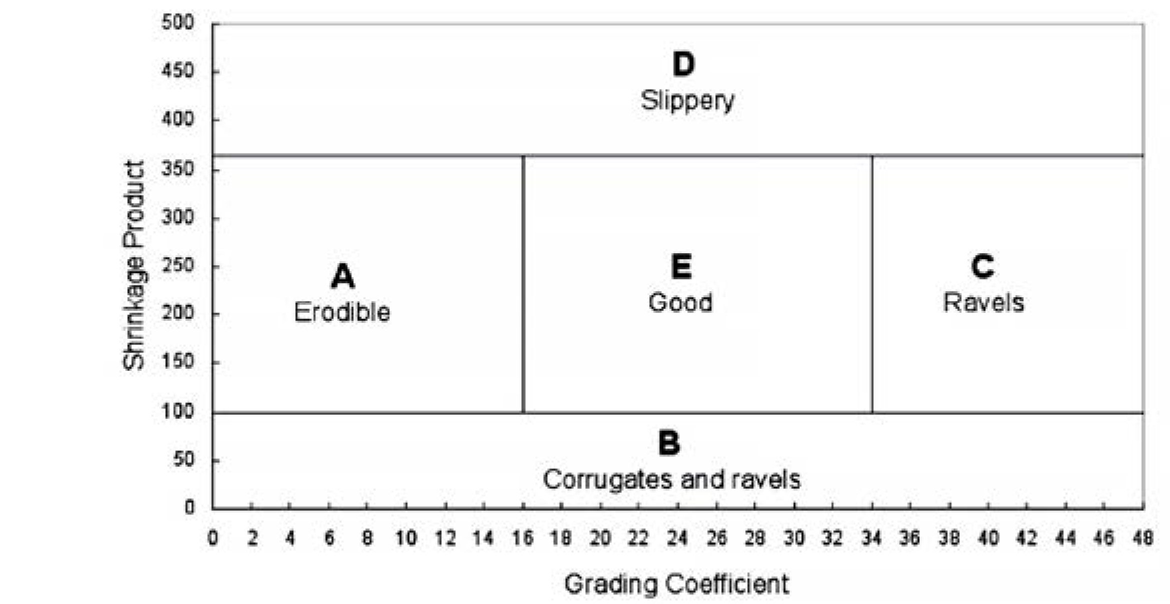
f. Use the guidance below to make the design and investment decisions:
   i. If alternative materials can be sourced look for alternative sources whose properties of GM and PP plot in a better material quality zone, Figure 14.1.
   ii. Consider the traffic, regravelling, maintenance grading cycles, life cycle costs and available budget to determine whether the road needs upgrading to sealed road standard, or regravelling, and be sustainably maintained as an unpaved road standard.
   iii. If the decision is to regravel, provide the quality of material using GM and PP.
   iv. If there are some materials with low plasticity and others with high plasticity, consider blending different materials in different proportions to meet the best performance Zone possible, Figure 14.1.
   v. Apply a thickness of the wearing course calculated from the rate of gravel loss per year and the desired regravelling cycle.

14.4.3 Design Check for Functionality

Performance of the wearing course in terms of the rate of gravel loss and roughness progression is the most important aspect of the design of unpaved roads. However, comfort and safety also play an important role. The selected materials for regravelling (overlay) must be checked on the functionality criteria.

Use the chart in Figure 14.5 to check functionality criteria regarding slipperiness, dust, corrugations, etc.

Figure 14.5 Functionality Specification for Wearing Course Material



14.5 Use of Do-Nou Technology

This section covers the application of Do-Nou technology in rehabilitating road sections that are failing due to weak substrate.

The situations where Do-Nou technology can be applied include:

1. Sections where existing soils and ground are weak – CBR < 3 %
2. Sections prone to flooding where embankments should be increased and strong materials are not available at economical distances.
3. Sections that are waterlogged, Figure 14.6.

Figure 14.6 Illustration of Application of Do-Nou Technology to Waterlogged Road Section

Before Dou-nou intervention

After Do-Nou intervention

14.5.1 Principles of Do-Nou Technology

The Dou-nou technology involves the use of synthetic bags filled with soils/gravel to construct pavement layers over weak subgrade, Figure 14.7. A typical pavement structure is shown in Figure 14.8.

Figure 14.7 Typical Do-Nou Cross Section for Improvement of Soft Ground

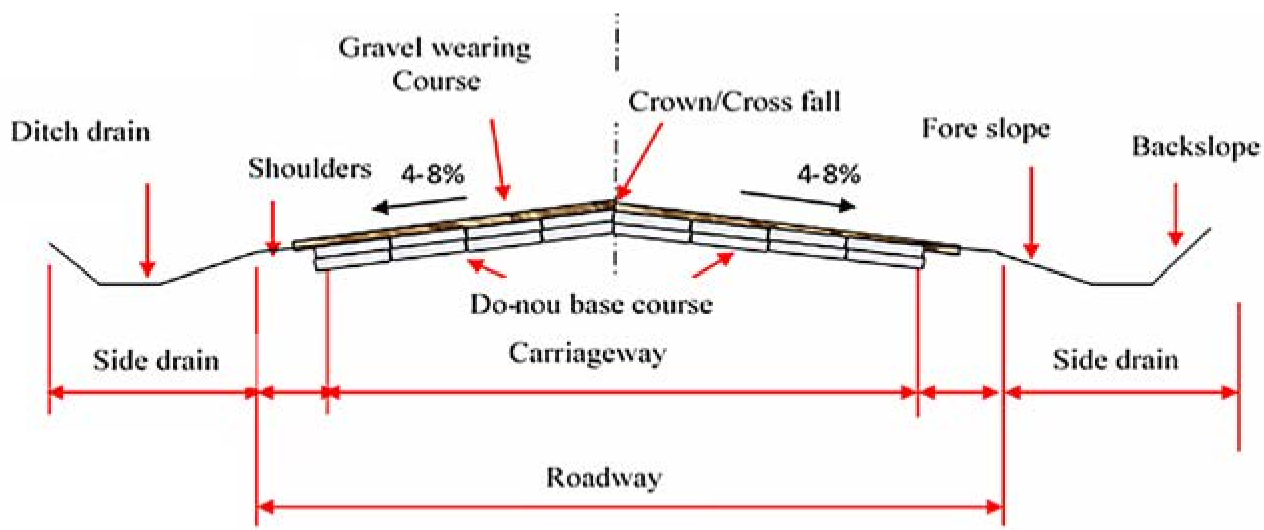
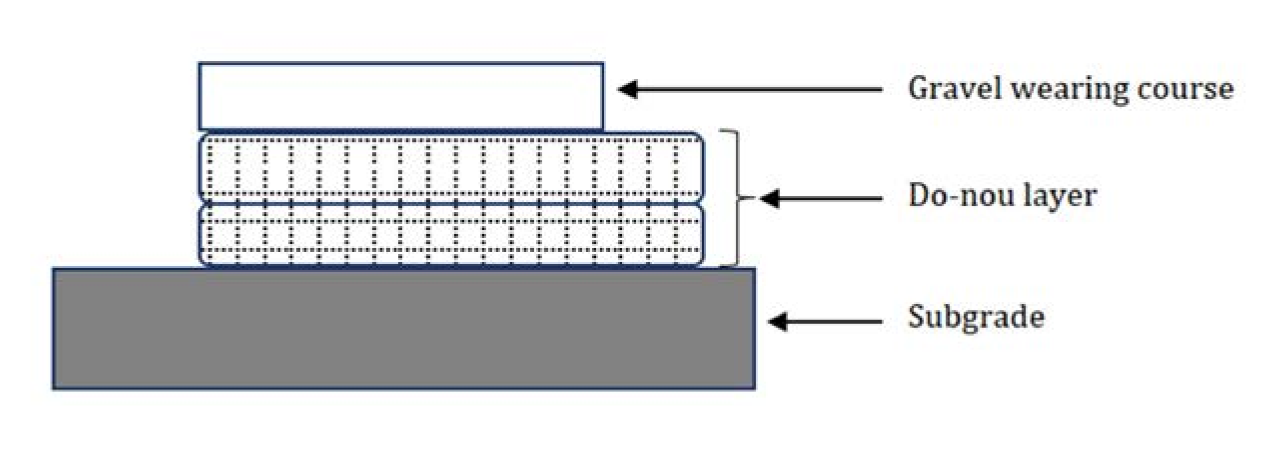


Figure 14.8 Basic Do-Nou Pavement Structure



The key principles of Do-Nou technology are detailed below:

1. Compaction aid – The bags provide confinement to the materials in the bag which aids compaction
2. Lateral support – The confinement significantly improves the overall stability of the pavement layer made out of the bags.
3. Improved pavement strength – The bags behave as a single unit (technically a single particle) with a large surface area leading to the following positive engineering effects:
   a. The pressure on the underlying weak materials/layer is greatly reduced due to the large surface area of the bag compared to the surface area of the individual particles of the soils/gravel acting individually.
   b. The bags provide reinforcement, and laterally confined compressive strength, which is significantly higher than the unconfined compressive strength in the case of unbound and unconfined material. This can be quantified through triaxial tests in the laboratory.
4. Retention of fines – the synthetic fabric retains fines inside the bags, improving durability and strength, especially in sections prone to flooding.
5. Erosion prevention – erosion is prevented, and the bags can be used as flood defences.

14.5.2 Strength Specifications for Do-Nou Bags

The properties of the geosynthetic material for the bags are critical for performance and details are provided in Table 14.4.

Table 14.4 Properties of Geosynthetic Materials For The Do-Nou Bags

Geosynthetic Material	Measure Minimum Tensile Strength (kN/m)	Max. Elongation (%)	Minimum Tensile Strength	Number of Strands Per Inch
Polypropylene (PP)	6.6	15	57	9-10
Polyethene (PE)	12	15		
Polyester (PY)	20	15		

14.5.3 Dimension of Bags and Strings

The dimensions of the bags are given in Table 14.5.

Table 14.5 General Dimensions of the Do-Nou Bags

Material	Dimensions	Specifications (mm)
Do-Nou bag	Length	600
	Width	450
Tying string	Length	350-400
	Diameter/thickness	5-8

14.5.4 Specifications for Do-Nou Materials

The general specifications for Do-Nou materials are given in Table 14.6 and Table 14.7.

Table 14.6 General Specifications For Do-Nou Materials

Material Parameter	Material Specifications	
	G8	G15
Maximum particle size (mm)	40	40
Percentage passing on 0.075 mm BS sieve (%)	-	Max. 50
Uniformity coefficient (%)	-	Min. 5
Swell (%)	Max. 1	Max. 1
Plasticity index (%)	Max. 30	Max. 30
Plasticity Modulus (PM)	-	Max. 2500
Organic matter (%)	Max. 2	Max. 2
CBR at 100% MDD AASHTO T99 and 4 days soaking	Min. 8	Min. 8
Minimum dry density as percentage of MDD AASHTO T99(%)	100	100
Compaction moisture content as percentage of OMC (AASHTO T99)	75-100	75-100
Minimum thickness of compact Layer (mm)	100	100
Maximum thickness of compacted layer (mm)	125	125

Table 14.7 Selection Criteria for Bag Fill

Type No.	Do-Nou Bag Geosynthetic Material	Do-Nou Bag Fill Materials	Do-Nou Gravel Cover
PP-1	Polypropylene	Natural material of CBR ≥ 8% (G8) Clay and sands	Natural gravel of CBR ≥ 20%
PP-2	Polypropylene	Natural material of CBR ≥ 15% (G15) Sands and gravels	Natural gravel of CBR ≥ 20%
PE-1	Polyethylene	Natural material of CBR ≥ 8% (G8) Clay and sands	Natural gravel of CBR ≥ 20%
PE-2	Polyethylene	Natural material of CBR ≥ 15% (G15) Sands and gravels	Natural gravel of CBR ≥ 20%
PY-1	Polyester	Natural material of CBR ≥ 8% (G8) Clay and sands	Natural gravel of CBR ≥ 20%
PY-2	Polyester	Natural material of CBR ≥ 15% (G15) Sands and gravels	Natural gravel of CBR ≥ 20%

14.5.5 Advantages and Limitations of Do-Nou Technology

Do-Nou technology has its advantages and limitations that the design engineer should consider (Table 14.8).

Table 14.8 Advantages and Disadvantages of Do-Nou Technology

Advantages		Limitations	
1.	Relatively simple	**1.**	It is cumbersome
2.	Geosynthetic bags are readily available and cheap	**2.**	It is suitable for small sections
3.	Technology is highly effective for low-volume roads	**3.**	The geosynthetic bags deteriorate due to weather elements including UV radiation. Exposure is detrimental to the durability of the bags
4.	Promotes the application of labour-based technology thus creating employment for local communities	**4.**	Not suitable for high-volume roads

15 Pavement Maintenance

15.1 General

This section provides guidance to the road agencies of Kenya on the maintenance of the existing road pavements in an efficient manner to ensure consistent standards, economical road maintenance and delivery of high levels of service.

Road infrastructure is one of the fundamental tools required to improve the socio-economic performance of a nation. The maintenance of roads to ensure pavement longevity, safety, good riding quality and comfort for the road users after construction cannot be emphasised enough.

The ultimate purpose of pavement maintenance is to rectify all or part of the defects identified on a specific pavement section to slow down deterioration and prolong the pavement life. Various engineering methods and techniques are employed to investigate and recommend a suitable treatment.

The techniques, methods and standards used in surveys of both the surface and structural condition of the pavements are detailed in RDM 5.1. This chapter complements the information given in RDM 5.1 by providing a brief of the defects and maintenance interventions necessary to mitigate such defects partially or completely. The maintenance of both the flexible and rigid pavements is covered in detail in Section 15.3 and Section 15.4 respectively.

Below is an outline of road maintenance classification:

1. **Routine Maintenance**

   Operations are required to be carried out once or more per year on a section of road. These operations are typically small-scale or simple, but widely dispersed, and require skilled or unskilled manpower. The need for some of these can be estimated and planned on a regular basis e.g. vegetation control.

2. **Periodic Maintenance**

   Operations that are occasionally required on a section of road after a medium-term period usually a number of years. They are normally large-scale and require specialist equipment and skilled resources to implement, and normally require the temporary deployment of those resources on the road section. These operations require specific identification and planning for implementation, and often require design and a significant budget allocation.

3. **Emergency (Urgent) Maintenance**

   Certain unforeseen situations necessitating remedial action to be taken as soon as possible e.g. flood damage, slips, etc. The intensity of investigation, planning and repair costs varies depending on the extent and severity of the damage.

   This manual has taken account of the basic conditions in Kenya. Engineers and inspectors in each region should consider the details of the road maintenance system based on this manual and the characteristic conditions in their region. Engineers and inspectors should also refer to existing handbooks and manuals such as:

   a. International Road Maintenance Handbook, Volume I, II, III, IV, PIARC, 1994

   b. Road Maintenance Handbook, Volume I, II, III, IV, UN, 1982

   c. Road maintenance Manual, RD of MORPW, 1992

   This manual was considered by a Working Group that consisted of members of the JICA Study Team together with engineers from the Ministry of Roads (MOR), the Ministry of Local Government (MOLG), Kenya Wildlife Service (KWS), and Nairobi City Council. The manual which was first published in 200I in three volumes have been reprinted as one volume.

15.2 Identification, Causes and Treatment of Visual Distress

Pavement maintenance classifications discussed above must follow the three-step procedure shown below:

1. Inspections to identify defects.
2. Evaluation to determine the root cause and selection of applicable repair methods.
3. Implementation of selected intervention methods and procedures.
    a. **Part I**; Inspection.
    This section describes the inspection methods as follows:
        i. Populating Inspection sheets.
        ii. Defect descriptions.
        iii. Frequency of inspection.
        iv. Safety procedures during inspections.
    b. **Part II**; Evaluation.
    This part describes the methods of evaluating defects and the selection methods for repair works. Part II includes the following:
        i. Evaluation standards.
        ii. Selection methods for execution of maintenance work.
    c. **Part III**; Execution.
    This part describes the methods for repair works, cleaning and clearing based on the evaluation results normally covering the following:
        i. Contents of each execution method.
        ii. Safety method during the execution of the works.

15.3 Maintenance of Flexible pavements

To successful implement maintenance works on flexible pavements, a thorough engineering assessment needs to be conducted in order to get an in-depth understanding of the existing pavement design, extent of deterioration, possible causes before determining the best solution.

Defects identified during engineering assessment of flexible pavements are divided into three categories:

1. Surfacing distress.
2. Structural distress.
3. Functional distress.

The principal modes of pavement distress of flexible pavements covered in this manual are as shown in Table 15.1.

A general depiction of the defects discussed above is shown in Figure 15.1, Figure 15.2 and Figure 15.3. These defects are described in the following section of this chapter, together with a photograph and a listing of the likely causes of the defect and the recommended treatments.

The causes of these defects can be categorised into three groupings:

1. Structural.
2. Environmental.
3. Construction quality (workmanship and materials).

In many cases, there may be more than one apparent cause which may makes it difficult to identify the primary cause of the defect.

Table 15.1 Summary of Road Defects Categories Bituminous Paved Roads

Item	Defect category	Pavement layer	Defects
1	Surfacing	Surfacing	• Surfacing cracks • Heaving/shoving, • Surface failure / delamination • Surfacing texture and voids, • Aggregate loss, stripping/fretting, • Binder condition, • Bleeding, • Surface patching, • Glazing, • Waving,
2	Structural	Pavement Layers	• Cracks (block, longitudinal, transverse and crocodile) • Cracks (block, longitudinal, transverse and crocodile) • Pumping, • Deformation, • Failure/potholes and patching.
3.1	Functional	Surfacing / Road shoulder	• Surface texture / roughness
3.2			• Bleeding • Polished aggregates (skid resistance) • Obstructions

Figure 15.1 Typical Deformation Defects In Flexible Pavements

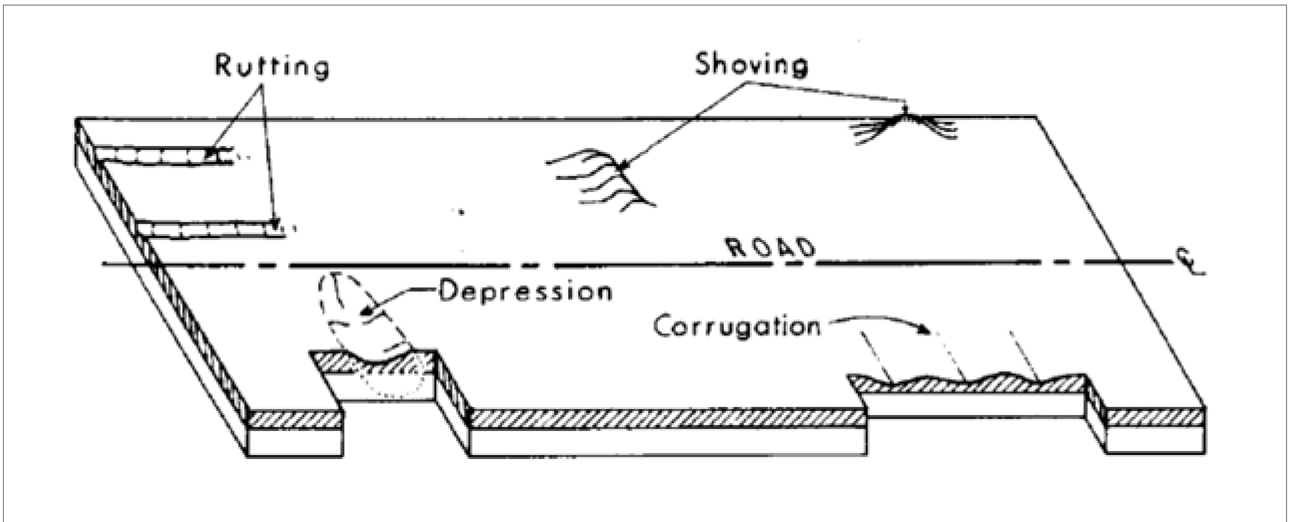
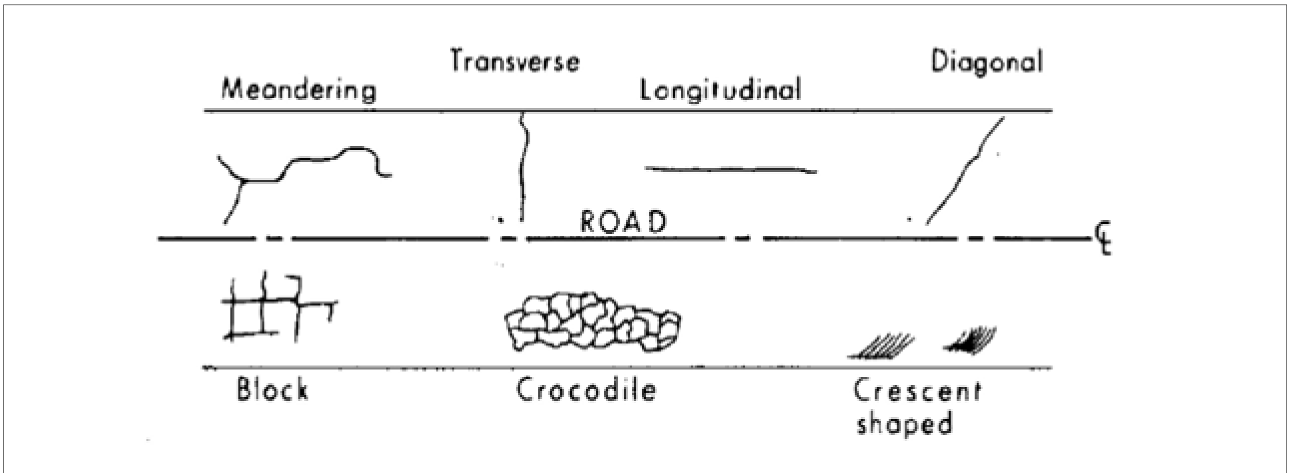
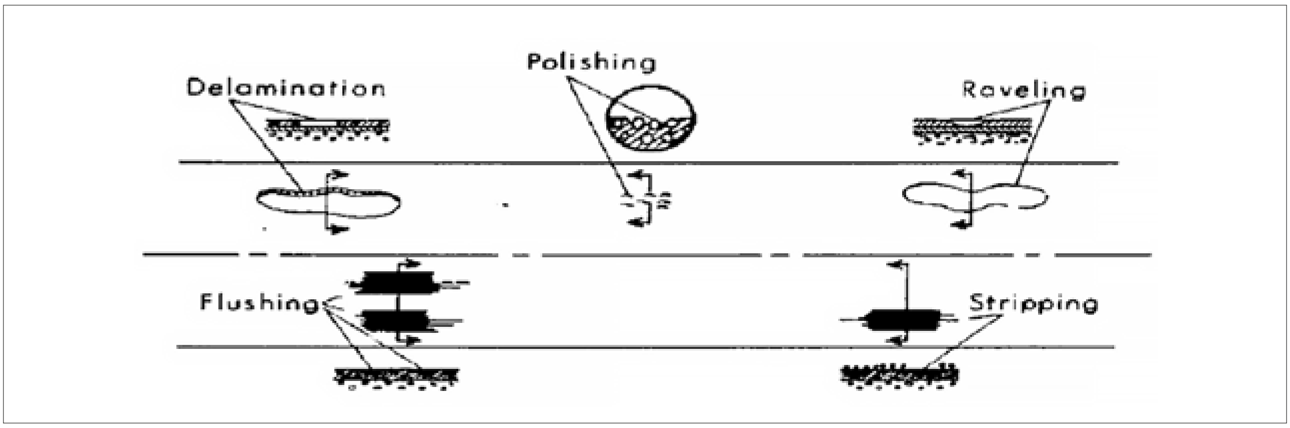


Figure 15.2 Typical Cracking on Flexible Pavements



1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
Pavement Maintenance

Figure 15.3 Typical Surface Distress Defects on Flexible Pavements



The following section of this document gives details on each defect summarised above. Each defect is explained in detailed tables providing the following information:

1. **Defect name and reference number**

   The table number and defect name are given as a subheading, placed at the top of each table for ease of identification of the defect. Each defect is discussed in a separate table for easy referencing.

2. **Description and definition of each defect**

   The Definition of Defects sheets have been developed to provide assessors across Kenya with a common understanding of the defects. Each defect has a single sheet which has several elements. The description gives information as is necessary to identify the defect on-site and to distinguish it from other defects.

3. **Defect Rating**

   This section of the table describes the extent of damage, which is key when determining the intervention methods.

4. **Indicative Picture (RDM 5.1)**

   A typical photo of the defect to aid the assessor classify the defects correctly and to eliminate ambiguity that comes with defects that are relatively similar visually.

5. **Effect**

   This gives a summary of what might happen if this defect is not repaired. This helps staff to appreciate the potential deterioration of the pavement

6. **Possible Causes**

   The possible causes are provided to help the Assessor to identify the causes or causes of this defect. This is important because different causes may require different actions for repair. This is particularly so in the case of water entering the pavement, which may require drainage works or maintenance to be conducted in addition to the pavement repairs.

7. **Interventions**

   The possible solutions are based on failure criteria associated with visible distress. Further confirmatory tests will need to be done especially where there is an indication of pavement distress.

15.3.1 Engineering Assessment - Surfacing

The existing (current) surfacing types and materials (Table 15.2) should be recorded. For ease referencing, the various surfacing types are assigned a reference code or notation to eliminate ambiguity.

Table 15.2 Surfacing Types

Item	Surfacing Type	Description	Reference Code/ Abbreviation
1	**Asphalt**	Continuously graded.	AC
		Gap graded.	AG
		Semi gap graded.	AS
2	**Surface Treatment**	Open graded.	AO
		Single seal	S1
		Multiple seal	SM
3	**Seals**	Sand Seal	SS1
		Cape Seal (Single seal & slurry seal)	CS
		Slurry Seal	SS2

15.3.1.1 Engineering Assessment of the Existing Surfacing

The surfacing types discussed above can best be identified by considering the attributes discussed below:

1. Surfacing texture

The texture of surfacing is defined by the depth of aggregate on the surface of the road and the quality of the binder. The surface texture is a key component that defines the skid resistance of a paved road. A distinction is made on the extent of deterioration using the Mean Profile Depth referred to herein as the MPD. For Engineering assessment purposes, the surface texture can be classified as:

a. Very Fine

Texture MPD < 0.4mm and **Fine** 0.4mm < MPD < 0.6mm: surfacing appears smooth and no visible coarse aggregate.

b. Medium

Coarse aggregate surrounded by fine aggregate resulting surfacing appearing smooth.

c. Course

MPD> 1.2mm: Surfacing has visible coarse aggregate.

d. Varying

Entails various texture classifications within a localised cross-section i.e., the texture along the wheel path appears smooth compared to other portions of the cross section. This can only be useful where there is a significantly varying surface treatment solution.

2. Surfacing voids

The aggregate size and binder quality influence the interconnected surface voids. The number of voids visible on a section of the surface informs the Assessor of the surfacing's affinity for a diluted emulsion, meaning the greater the susceptibility of surfacing to absorb the more diluted emulsion, the more the surface voids. Surface voids can therefore be categorised into the following four distinct groups:

a. Low:

There is evidence of bleeding signified by a dense surfacing with no voids.

b. Medium:

Few visible voids.

c. High:

Many visible voids.

d. Varying:

The voids vary within a single cross-section of the same type of surfacing. It should be noted that if the seal types vary, the voids cannot subsequently be considered as varying.

15.3.1.2 Surface Cracking

Surface cracking is normally limited to the surfacing layer / bituminous layer only. Surface cracks are common and easy to identify in dense and fine-textured surfaces. Surfacing cracks are usually spread across the carriageway and not restricted to the wheel path. This is used to distinguish between severe surface cracks and structural (crocodile) cracks depending on the crack width or depth (severity of the deterioration).

Defect Description

Faint Shrinkage Cracks

Small/faint cracks which start off with a star shaped pattern which develops blocks which resemble the alligator or crocodile cracking pattern.

Closely Spaced Transverse Cracks

These cracks start off as closely spaced fine transverse cracks which develop into distinct cracks eventually forming a crocodile crack pattern.

Aging surface treatments normally exhibit a crocodile crack pattern; therefore, it should be noted that this does not signify pavement failure. In such instances it is diligent to consider other less damaged sections to deduce the correct category of the crack.

Effects if neglected

1. Progressive break-away of chippings resulting in the surfacing becoming slippery, permeable and worn out by traffic.
2. If deterioration is severe, it causes uneven ride and potholes which becomes a safety risk for the motorists.

Interventions

1. Crack sealing or use of geo-grid.
2. Fog Spray.
3. Rejuvenation.
4. Strain Alleviating Membrane (SAM) seals, reinforced seals, ultra-thin overlays.
5. Strain Alleviating Members Interlayers (SAMI) or geotextile seal plus asphalt overlay.
6. Cold plane and overlay.
7. In situ asphalt recycling and overlay.

15.3.1.3 Ravelling / Aggregate Loss

Defect Description

Ravelling also known as aggregate loss is the dislodging of coarse aggregate from a seal leaving the binder exposed to wheel contact. The process starts off as loss of individual stones and eventually the complete loss of stones in a localised area. This defect commonly occurs soon after construction or during the first cold dry seasons. Aggregate loss is as active if the aggregate loss is continuing and as non-active if the loss of aggregate has stopped. In Asphalt aggregate loss first causes disintegration which is followed by cracking and eventually develops into a pothole. In thin surfacing types, aggregate loss results in the exposure of the pavement layers.

Effects if neglected

1. Developing of cracks which will expose the pavement structure to moisture ingress resulting in the formation of potholes.

Interventions

1. Enrichment or rejuvenation of binder – only where aggregate loss is limited to few stones
2. Reseal.

15.3.1.4 Surface Failure / Delamination

Defect Description

Loss of binder and aggregate in surfacing causing clear delineation between the wearing course and the underlying layer or original surface of a resurfaced road. This is a surface defect only if it has not caused significant distress on the underlying layer otherwise, it should be considered as a structural defect.

Interventions

1. Crack filing
2. Fog Spray
3. Rejuvenation

15.3.1.5 Surface Deformation / Shoving

Defect Description

This defect manifests as the budging of the pavement surfacing layer as a result of plastic flow. Surface deformation is common in high-shear stress zones on road sections where braking and acceleration of heavy vehicles are dominant. This type of defect will occur in either the transverse or longitudinal direction. The deformation in the transverse direction should not be confused with rutting.

Effects if neglected

1. Increased vehicle and maintenance costs
2. Reduced road safety and increases the risk of traffic accidents due to poor riding quality/discomfort.
3. Slows down traffic resulting in increased trip duration and driver discomfort.

Interventions

1. Re-surfacing
2. Sectional//Partial reconstruction of damaged pavement layers. If the subgrade material is poor, then the ideal stabilisation agents must be applied to improve pavement material properties.
3. Heavy patching with pavement layer works repairs.
4. Provide sub-surface drainage to lower the water table.
5. In-situ asphalt recycling (inlay) or Asphalt or granular overlays
6. Cold planing of unsound material and replacement with adequate material
7. Where due to deficiency of unsealed shoulder, re-gravel shoulder.

15.3.1.6 Surface Patching

Defect Description

Repaired sections of pavement usually ranging from less than 1m up to several meters equivalent to half or full pavement width. This is minor patching with indistinct edges (Edges are not cut into distinct shapes as done with structural patching.). The type, shape, size of and frequency of patching provides information on the serviceability of the existing surfacing. These characteristics distinguish surfacing patching from structural patching. Surfacing patches typically occur outside the wheel path whereas structural patches are normally located within the wheel paths.

Effects if neglected

1. Exposure of pavement structure to moisture ingress resulting in under washing of pavement layers which accelerates the formation of potholes which ends up as premature pavement failure.
2. Uncomfortable ride quality.
3. Road safety risk to the motorists and speed reduction.

Interventions

1. Crack sealing
2. Skin patching
3. Overlay

15.3.1.7 Binder Condition

Defect Description

This section considers how the elastic and viscous properties of bitumen binders change with temperature and time, thereby resulting in the development of defects in the surfacing layer. During engineering assessments, it is key to note the resultant visible effects of these changes in order to determine the relevant repair methods.

In thick surfacing, deterioration starts with the loss of the volatile oils and aromatics and oxidation in the surface. The products from the oxidation process dissolve in water and tend to shrink, which then increases with every rainfall event triggering continued oxidation which penetrates deeper into the binder film.

To assess the extent of deterioration of the binder, the assessor should remove pieces of aggregate within the wheel path to check the binder for dryness. If the binder is still in good condition, it will show a bright black colour and be dull if the binder is dry. It is important to avoid conducting this test when the temperature is below 20°C because some binders appear dry if the temperature is low.

Effects if neglected

1. Surfacing cracks will start developing and moisture ingress will damage the underlying pavement layers.

Interventions

1. Fog spray
2. Surface dressing
3. Inlays or overlays were applicable

15.3.1.8 Bleeding

Defect Description

Bleeding occurs when excess bituminous binder fills the aggregate voids and expands onto the pavement surface during hot weather. The process is irreversible meaning in cold weather, the binder is absorbed back into the voids but accumulates on the surfacing creating a shiny black surface film.

Effects if neglected

1. Poor skid resistance on affected road sections i.e., roads become slippery when wet or hot.
2. Separation and breakaway of the surface layer under traffic load..

Interventions

1. Reseal
2. Solvent treatment plus additional aggregate- for flushed sprayed seals
3. Cold planing, scabbing, and grooving - for fatty asphalt
4. Asphalt overlay

15.3.2 Engineering Assessment – Structural.

This part of the manual provides guidelines on how to assess the condition of the existing pavement structure in order to identify potential or signs of distress. Visible defects give an idea on the level of deterioration in the strength of the pavement structure typically caused by traffic loading, climate, environmental impact, surfacing defects, pavement material quality etc. This information coupled with information of the condition of surfacing assist in deciding the type and extent of maintenance required.

A detailed outline of all the defects related to the deterioration or failure of pavement structural layers is discussed in this section:

15.3.2.1 Transverse Cracking

Defect Description

Unconnected cracks running across o/ perpendicular to the direction of travel. Transverse cracks are related to shrinkage in cement-stabilised bases or subbases o, temperature-related fatigue and climate. They can also be a sign of poor compaction in the pavement structure along the edges of the section where after the reinstatement of the pavement layers following the installation of services across the road. The assessor should be able to distinguish surface cracks and edge defects from transverse cracks.

Effects if neglected

1. Allows moisture ingress into underlying pavement layers resulting in softening and weakening of lower layers resulting in premature pavement failure.

Interventions

1. Crack seal.
2. Strain Alleviating Membrane (SAM) seals, reinforced seals, ultra-thin overlays.
3. Strain Alleviating Membrane interlayer (SAMI) or geotextile seal plus asphalt overlay.
4. Cold planing and overlay.
5. In situ asphalt recycling.

15.3.2.2 Longitudinal Cracking

Defect Description

Cracks that usually start forming in the wheel path at its early development stage and spread across the entire cross-section, running parallel to the direction of travel with occurrence of limited branching in some cases. Deterioration is caused by a lack of maintenance and traffic loading. The assessor should ably distinguish between surface cracks and edge defects from transverse cracks.

Effects if neglected

1. Deterioration of pavement structural layer due to moisture ingress through the cracks.

Interventions

1. Drainage improvements
2. Sealing of shoulders
3. Crack filling
4. Cold planing and overlay
5. Heavy patching
6. Reconstruction.

15.3.2.3 Block Cracking

Defect Description

Interconnected cracks forming a series of definite block patterns. Typically distributed over a large area of pavement.

In the initial stages of their development, do not necessarily signify pavement deterioration but potential of deterioration rather. However, with further exposure to traffic action, secondary cracks are formed, which eventually leads to severe destress.

Effects if neglected

1. Deterioration of pavement structural layer due to moisture ingress through the cracks.

Interventions

1. Crack filling.
2. Strain Alleviating Membrane (SAM) seals, reinforced seals, ultra-thin overlays.
3. Strain Alleviating Members Interlayers (SAMI) or geotextile seal plus asphalt overlay.
4. Cold plane and overlay.
5. In situ asphalt recycling and overlay.

15.3.2.4 Crocodile Cracking

Defect Description

Interconnected or interlaced cracks that usually form within the wheel path indicating failure of surfacing or base layer. Crocodile cracks normally signify the end of the pavement design life. Crocodile cracking is often confined to the wheel paths and may have a noticeable longitudinal grain. The presence of crocodile cracking usually signifies that the surfacing has reached the end of its design life.

Effects if neglected

1. Deterioration of pavement structural layer due to moisture ingress through the cracks.

Interventions

1. Alleviating Membrane (SAM) seals, reinforced seals, ultra-thin overlays.
2. Strain Alleviating Membrane Interlayer (SAMI) or geotextile seal plus thin asphalt overlay.
3. Cold planing and thin overlay.
4. In situ asphalt recycling.
5. Drainage improvements in combination with another treatment such as an overlay.
6. Strain alleviating Membrane interlayer (SAMI) plus asphalt overlay.
7. In situ asphalt recycling plus overlay.
8. Cold planing plus overlay.
9. In situ stabilisation.
10. Heavy patching.
11. Reconstruction.

15.3.2.5 Diagonal Cracks

Defect Description

Unconnected crack running diagonally across the pavement.

Effects if neglected

1. Deterioration of pavement structural layer due to moisture ingress through the cracks.

Interventions

1. Crack seal.
2. Strain Alleviating Membrane (SAM) seals, reinforced seals, ultra-thin overlays.
3. Strain Alleviating Membrane Interlayer (SAMI) or geotextile seal plus asphalt overlay.
4. Cold planing and overlay.
5. In-situ asphalt recycling and overlay.

15.3.2.6 Meandering Cracks

Defect Description

Unconnected irregular crack, varying in line and direction: This usually occurs singly.

Effects if neglected

1. Deterioration of pavement structural layer due to moisture ingress through the cracks.

Interventions

1. Crack filling.
2. Strain Alleviating Membrane (SAM) seals, reinforced seals, and ultra-thin overlays.
3. Strain Alleviating Membrane interlayer (SAMI) or geotextile seal plus asphalt overlay.
4. Cold planing and overlay.
5. In situ asphalt recycling and overlay.
6. Remove trees.

15.3.2.7 Crest-Shaped Cracks

Defect Description

Unconnected irregular parabolic crack, varying in direction. This usually occurs singly.

Effects if neglected

1. Deterioration of pavement structural layer due to moisture ingress through the cracks.

Interventions

1. Crack filling.
2. Cold planing and overlay.
3. In-situ asphalt recycling and overlay.
4. Overlay.
5. Heavy patching.
6. Reconstruction.

15.3.2.8 Potholes

Defect Description

Loss of material in the wearing course and underlying pavement layers mostly in the wheel path. Potholes signify structural failure induced by traffic action. Potholes develop from cracking or severe loss of aggregate and therefore they are a secondary form of distress which starts from the top of the pavement downwards.

Effects if neglected

1. Allows water to enter the pavement causing softening and weakening of the pavement and lower layers. This may cause premature failure of the pavement.
2. If left unrepaired, can rapidly expand the extent of the damage.
3. Creates poor ride quality for the motorist and may reduce traffic speed.
4. If large can cause damage to vehicle wheels and tyres.
5. Can create an accident risk.

Interventions

1. Normally repaired by routine maintenance techniques.
2. Reseal.
3. Patching and overlay.

15.3.2.9 Pumping

Defect Description

This happens when a mix of water and fine aggregates are ejected from the underlying pavement layers onto the surface through existing cracks under traffic load. This normally happens after recent rains or in a high-water table environment.

Effects if neglected

1. Popping off the surfacing layer.
2. Moisture damage resulting in potholing.

Interventions

1. Providing adequate sub-surface drainage mainly and surface drainage in some instances.
2. Full-depth patching in the affected sections and re-construction of affected layers.

15.3.2.10 Deformation – Rutting

Deformation defects are changes in the road surface profile in which the current surface is below or above the original finished pavement level.

Defect Description

Rutting is deformation in the wheel path attributed to traffic action on weak pavement and subgrade material resulting in plastic shear deformation of upper layers. Sharp and narrow ruts are formed when the distress is confined to the upper pavement layers whereas Wider and even ruts are a sign of distress in the lower pavement layers.

Effects if neglected

If water penetrates the pavement structure, then there will be a rapid increase in the degree of rutting often leading to cracking and break-up of the pavement.

Causes a reduction in serviceability, reduces vehicle travel speeds and compromises driver comfort and poses road safety risks.

Interventions

Non-structural Treatments

1. In-place asphalt recycling.
2. Cold planing to remove high points.
3. Cold overlays, slurry seals (provided rut depth 15 mm) and micro surfacing.
4. Thin asphalt surfacing (non-structural).
5. Ultra-thin overlays-provided rut depth ≤ 2.5 x mix size.
6. Rip, reshape and reseal - appropriate for granular pavements.

Structural Treatments

1. Drainage improvements.
2. Asphalt overlay or granular re-sheet.
3. Deep lift asphalt.
4. Partial reconstruction and asphalt overlay.
5. In situ stabilisation.
6. Heavy patching.
7. Total reconstruction.

1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
Pavement Maintenance

15.3.2.11 Deformation – Corrugations

Defect Description

Transverse undulations in the pavement surface or base, are most associated with spray seal or unsealed pavements, but can occur in thin asphalt-surfaced roads too.

Wavelengths of undulations can range between 0.3 and 2 metres.

Effects if neglected

1. If water penetrates the pavement structure, then there will be a rapid increase in the degree of rutting often leading to cracking and break-up of the pavement.
2. Causes reduction in serviceability, reduces vehicle travel speeds, compromises driver comfort and poses road safety risks.

Interventions

1. Remove and replace the base and reseal or overlay.
2. In situ stabilisation.
3. Cold plane and overlay asphalt surfaced pavements.

15.3.2.12 Structural Patching

Defect Description

Structural patching gives information on the historic distress. The type and average size of the patch. give the assessor an idea of the extent, type and severity of the distress that was previously repaired.

Effects if neglected

1. The pavement layers will be exposed to moisture ingress if the edges of the patches are not properly sealed. This results in the formation of potholes.

Interventions

1. Normally repaired by routine maintenance techniques
2. Reseal
3. Patching and overlay.

15.3.2.13 Failures

Defect Description

Structural failures manifest as mounds along the road edge next to the depression on the wheel tracks. The resultant lateral displacement is evidence of loss of shear strength in the pavement layers mostly the base layer due to traffic action.

Effects if neglected

1. Continued deterioration of pavement material resulting in potholes.
2. The trip durations will be increased and the road will be unsafe for use.

Interventions

1. If minor, repair cracks and re-surface
2. Milling and inlay
3. If severe, rehabilitate damaged pavement layers.

15.3.3 Engineering Assessment - Functional

Over and above the need to assess the structural performance of the pavement, defects that affect the perception of the road user which include speed of travel, comfort and safety need to be evaluated

The following section discusses the functional defects prevalent in flexible pavements:

15.3.3.1 Riding Quality / Roughness

Defect Description

Roughness is defined as the condition of the road profile in relation to rutting variance and longitudinal deformation in the wheel path. The road user's perception of the road condition whilst driving is based on their experience measured by the smoothness of the surface and the riding comfort emanating from irregularities in the pavement surface.

Effects if neglected

1. Poor riding quality for motorists.
2. High vehicle operation and maintenance costs. (Fuel and repair costs respectively).
3. Disruption in traffic flow due to speed reduction.
4. Continued deterioration of the pavement making repairs more expensive with time.
5. Fluctuation in pavement stresses.

Interventions

1. Mill and inlay.
2. Resurfacing.
3. Rehabilitation.

15.3.3.2 Skid Resistance

Defect Description

This is the measure of the road surface's slipperiness in wet conditions. This condition is determined by the surface texture especially the visual coarseness of the surface in relation to macro texture depth, voids between the aggregate and roughness of surfacing aggregates to visually check if they are angular and rough, round and smooth.

Effects if neglected

1. Reduced road safety and potential for road accidents.

Interventions

1. Rolled in aggregate.
2. Resurfacing.

1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 Pavement Maintenance

15.3.3.3 Edge Defects

Defect Description

Edge defects most commonly occur in narrow surfaced roads where the outside wheel path is close to the surfaced edge.

Edge defects can be categorised as follows:

1. **Edge Break:**
   Breaking away of surfacing at the edge of the road due to lack of or poor unpaved shoulder maintenance.
2. **Short Transverse Cracks:**
   Cracks that start from the edge and migrate inwards.
3. **Drop Off:**
   A step (significant elevation difference) formed due to erosion of the gravel shoulder-wearing course.

Note:
Longitudinal cracks within 300 mm of the edge of the road should be classified as edge breaking.
Edge breaking eating into the wheel path can be equated to potholing.

Effects if neglected

1. Continued edge breaking leads to potholing and eventually the road becomes too narrow.
2. Exposes pavement layers to further deterioration due to exposure to environmental damage.

Interventions

1. Regular regravelling of the unpaved shoulder and adequately compacting the granular material and wearing course. This should be done just before and after the rainy season.
2. Repair surfacing at the edges of the road frequently.
3. Widen the surfaced road shoulder.
4. Reduce unsurfaced shoulder slopes.

15.3.3.4 Unpaved Shoulder Conditions

Defect Description

An unpaved shoulder works as an additional safe recovery space for a paved road. Significant edge drops can occur making it unsafe for motorists. Gullies can also form due to runoff from the carriageway. Unpaved shoulders' suitability for purpose is then compromised.

All road edges restrained by edge beams, lined drainage structures or kerbs are considered to have no shoulders because the full width would be used as the carriageway.

Effects if neglected

1. Continued edge breaking resulting in narrowing of the paved carriageway.
2. Unsafe road conditions.
3. Increase in vehicle maintenance costs.

Interventions

1. Re-gravelling and ensuring good compaction.
2. Repair surfaced edge and
3. Reduce unsurfaced shoulder slopes.

1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
Pavement Maintenance

15.3.3.5 Surface Drainage

Defect Description

Surface drainage is defined by the ability and rate at which the road geometry clears off water away from the surface during and after any rainfall event. The surface drainage can also be rated by its ability to keep away grit washed onto the road surface from the verges. This defect can easily be rated according to the extent of ponding especially within the wheel paths and the volume of water splashes when driving.

Effects if neglected

1. Moisture ingress into the pavement which will eventually cause premature pavement failure.
2. Reduced road safety which may cause accidents.

Interventions

1. Patching and Inlay where minor defects apply,
2. Asphalt overlay to correct irregular surfaces,
3. Reconstruct.

15.4 Maintenance of Rigid Pavements

15.4.1 General

This section covers the maintenance of rigid pavements. It provides guidance on the most common defects associated with rigid pavements, their identification and treatment. The treatments differ depending on the type of concrete and guidance is provided below on jointed and continuously reinforced concrete pavements.

15.4.2 Surface Defects and Maintenance of Jointed Rigid Pavements

Table 15.3 Surface Defects and Maintenance of Jointed Rigid Pavements

Item No.	Type of visual defect	Possible causes	Maintenance interventions
1	**Defective joint seals**	Adhesion/cohesion failure, age of sealant.	Reseal joints.
2	**Surface irregularities** (poor surface profile)	Bumps and/or depressions	Remove high areas by bump cutting (and retexture if necessary) ensuring that adequate cover is maintained above reinforcement.
3	**Surface scaling**	Concrete had inadequate cement content or entrained air. Vehicle fire or acid attack from spillage.	Retexture surfacing ensuring that adequate cover is maintained above reinforcement (ST) For small areas (typically ≤1 m2) thin-bonded repair. In severe cases replace the slab.
4	**Pop-outs** (Isolated loss of a small area of surface material)	An aggregate particle expands and fractures as a result of a physical action or a chemical reaction.	Carry out thin-bonded repair. Plug single particle pop-outs with resin mortar.
5	**Shallow joint spall** (Spall extends < 50 mm from joint and depth is < one-third of slab depth).	Weaker concrete, trafficking, failed repair, ingress of material into the joint groove, etc.	Widen joint sealant groove (maximum width 40 mm) - recut & seal joint. Thin-bonded repair

1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 Pavement Maintenance

Table 15.4 Structural Defects and Maintenance of Jointed Rigid Pavements

Item No.	Type of visual defect	Possible causes	Maintenance interventions
1	**Corner crack(s) (0.3 - 2 m long)**	Poor slab support and/or load transfer, dowel bar issues. Often starts as a single crack but can quickly develop to form multiple corner cracks or further cracks across corner segments. If larger than 2 m, consider as a diagonal crack. If less than 0.3 m, consider as a shallow/deep joint spall. Acute angles in non-rectangular slabs.	No immediate treatment if the crack is < 0.5 mm wide. Full-depth joint repair.
2	**Cracks around gully or utility cover**	Discontinuity and associated stresses caused by gully/utility cover within a slab, particularly at corners of the slab adjacent to gully/utility cover.	No immediate treatment if slab is reinforced and crack < 0.5 mm wide. Relocate gully/utility cover in verge and replace full or partial slab.
3	**Diagonal cracks** (Excluding corner cracks or cracks associated with gully/utility cover). *(Condition can be investigated by coring the crack and an FWD survey to evaluate the performance of the cracks).*	Differential settlement	No treatment initially but monitor crack. (e.g., reinforced concrete single crack < 0. 5 mm wide). For wider crack, form groove and seal. Full-depth repair. Replace the slab (this will depend upon the severity of the defect).
4	**Transverse cracks** (Condition can be investigated using an FWD survey to evaluate the performance of the cracks)	**At construction:** excessive slab length, late sawing of joint grooves, displaced bottom joint former or inadequate reinforcement lap. **Post construction:** poor or variable support (foundation), poor load transfer to an adjacent slab or joint lock-up (crack is usually near the transverse joint).	No treatment for cracks <0.5 mm wide. Monitor. (Epoxy treatment (crack width 0.5 to 1.5 mm). Full-depth repair for unreinforced slabs or reinforced slabs with corroded steel (Crack width >1.5 mm). Replace slab (crack width > 1.5 mm or multiple longitudinal & transverse cracks).
5	**Longitudinal cracks** (Condition can be investigated by coring the crack and an FWD survey to evaluate the performance of the cracks)	Constructed with excessive slab width, differential support across slab (e.g., by water penetration to the foundation), misaligned or locked-up dowel bar(s) or induced by the cumulative effects of traffic.	Monitor (if reinforced and crack <0.5 mm wide). Full-depth repair for unreinforced slabs or reinforced slabs with evidence of corroded steel (crack width >1.5 mm). Replace slab (crack width > 1.5 mm or multiple longitudinal & transverse cracks).

Item No.	Type of visual defect	Possible causes	Maintenance interventions
6	**Deep joint spall** (where spall extends more than 50 mm from edge and more than one-third of slab depth). (Condition can be investigated by coring the spall and an FWD survey to evaluate the performance of the joints)	Weakened foundation at the joint, dowel bar issues, weaker concrete, trafficking. (More prevalent in wheel paths), failed repair, etc.	Full-depth joint or slab end repair (full width).
7	**Punchouts**	Localised defect in reinforced concrete where multiple cracks have joined up. The fragments of concrete generally appear to have been 'punched' downwards.	Full-depth repair.
8	**Slab rocking / Slab settlement / Joint Stepping** (Where step > 3 mm between adjacent slabs) (Condition can be investigated by coring the joint to examine dowels & tie bars [after locating with GPR, metal detector, etc], DCP to examine the foundation and an FWD survey to evaluate the performance of the joints)	Reduced sub-base support (often due to water penetration) and/or corroded tie bars/dowel bars. Rocking (i.e., visible vertical movement of slab edge) may be observed particularly when trafficked by heavy vehicles. Rocking is often identified from mud-pumping stains from a joint.	Inject grout under slabs/slab lifting. Remove high areas by bump cutting and retexture if necessary, ensuring that adequate cover is maintained above reinforcement (ST). Improve under slab drainage where necessary prior to pavement maintenance being undertaken. Full-depth slab end repair Replace slab (and sub-base if required) as appropriate. Retrofit additional tie bars to adjacent lanes if required.
9	Compression failures (known as 'blow-ups'). These usually occur in jointed pavements during periods of prolonged hot weather.	Substantial build-up of detritus in joint grooves because of poor joint sealing and/or inadequate number of expansion joints formed during construction. This failure is more likely in jointed pavements greater than 8 years old with evidence of poor sealant condition/ build-up of detritus in joint grooves/ a significant number of temporary joint repairs or sealing grooves that cannot be sealed properly and/or at least two consecutive locked-up joints.	Repair or replace 'blown up' slabs (or slab ends) to full depth. To prevent future blow-ups, install new dowelled expansion joints in all lanes at 2 50 m centres over the relevant lengths.

Table 15.5 Structural Defects in Continuously Reinforced Concrete Pavements

Item No.	Type of visual defect	Possible causes	Maintenance interventions
1	**Normal** (transverse) cracks) ('Normal' cracks in CRCPs are exclusively transverse with no spalling or bifurcations, are ≤1 mm in width and are spaced at least 1 m apart)	It is normal for fine transverse thermal contraction/shrinkage cracks to develop every 1-4 m soon after construction. Over the next 4 years or so, further transverse cracks slowly develop between the wider-spaced cracks, resulting in a typical crack spacing of 1-2 m. These cracks are an inherent feature of continuously reinforced concrete.	No treatment
2	**Crazing**	The presence of any crazing (referred to as 'map cracking' in the CPMM) should be noted, particularly any containing white or cream powdery material that streaks over the surface after heavy rain as such a deposit may indicate an alkali-silica reaction.	Seek guidance from the overseeing organisation.
3	**Spalled cracks**	Weakened foundation at crack, weaker concrete, trafficking (more prevalent in wheel paths), failed repair.	Shallow spall - Thin-bonded repair. Monitor and record locations of spalling for future reference. Deep spall - Full-depth repair (one lane at a time to minimise stress in the pavement)
4	**Punchouts**	Localised defect in reinforced concrete where multiple cracks have joined up. The fragments of concrete generally appear to have been 'punched' downwards.	Full-depth repair
5	**Significant transverse cracks.** (Significant transverse cracks in CRCPs include any of the following: 1. Spacings of less than 1 m. 2. >1 mm wide. 3. Bifurcated. 4. Polygonal.	These can be caused by a design fault, a construction fault, poor quality concrete or a lack of foundation support. For cracks ≤1.5 mm wide, the reinforcement may not have yielded completely. If left untreated, water ingress can lead to corrosion and spalling. At wider cracks, the reinforcement has almost certainly yielded. In effect, the cracks are likely to be acting as either undowelled or untied joints	Isolated cracks (≤1.5 mm wide) may be sealed. Wide cracks (> 1.5 mm), or where significant defects are widespread, full depth repair.
6	Longitudinal cracks	Longitudinal cracks are not expected. Possible causes include shallow cover to reinforcement, slab design is too thin and/or poor foundation support. Longitudinal cracks may well deteriorate and develop further unless remedial action is taken. Where longitudinal and transverse cracks cross there is a risk of spalling particularly if they cross obliquely. When sealing cracks, a sawn groove is preferred because it is much more regular and can be made narrower.	Seal isolated cracks > 0.5 mm. Full-depth repair depending on severity.

16 References

1. National Highways UK/TRL, Design Manual for Roads and Bridges
2. Queensland Department of Transport and Main Roads – Australia, Pavement Design Manual
3. AASHTO Handbook for Pavement Design, Construction and Management.
4. AASHTO, 1993, AASHTO Guide for Design of Pavement Structures
5. SANRA, 2013, South African Pavement Engineering Manual, SAPEM
6. TRL/ANE, 2019, Rehabilitation Design Manual for Mozambique
7. Ministry of Roads -Kenya, 2010, The Road maintenance Manual.
8. MTRD, 2017, Pavement Design Guideline for Low Volume Sealed Roads
9. MTRD, April 1987, Minor Roads Programme Interim Technical Manual
10. IRC 1152014, Structural Evaluation and Strengthening of Flexible Pavements Using FWD
11. IRC 117-2015, Structural Evaluation of Rigid Pavements
12. MTRD, KEN 042, 2013 Research on Low-Cost Pavements - Field measurements data
13. MTRD, 2019, Pavement Evaluation Report of Dagoretti Corner - Karen Roundabout Road
14. AASHTO, 2002, American Association of State Highway and Transportation Officials Test Standards Manual
15. TRL, 2001, Road Note 18 (ORN18) A guide to the Pavement Evaluation and Maintenance of Bitumen-Surfaced Roads in Tropical and Sub-Tropical Countries
16. MTRD, 2012, Report No.1548- Bus Rapid Transit (BRT) facilities on Thika Superhighway corridor Pavement Testing Report
17. KeNHA, March 2023, Performance monitoring of Timboroa -Eldoret -Webuye-Malaba MTRD report No.1538
18. MTRD, LR886 - Performance of sections of the Nairobi to Mombasa Road in Kenya
19. Ministry of Roads Kenya, 1978, Road Design Manual Part V Pavement Rehabilitation and Overlay Design.

Appendix A:

Catalogues of Overlay Thickness Design based on Equivalent Modulus

ASPHALT CONCRETE TYPE II

OVERLAY DESIGN Catalogue No. 1 | **E = 2500 MPa**

Traffic x10^6	Equivalent modulus in MPa - Overlay thickness in mm													
	25	50	75	100	125	150	175	200	250	300	350	400	450	500
.3	133	103	81	66	54	44	37	31	30	30	30	30	30	30
.4	140	110	88	71	59	48	40	34	30	30	30	30	30	30
.5	146	116	93	76	63	52	44	37	30	30	30	30	30	30
.6	151	120	97	80	66	55	47	39	30	30	30	30	30	30
.7	156	125	101	83	69	58	49	42	30	30	30	30	30	30
.8	160	128	105	87	72	61	51	44	32	30	30	30	30	30
.9	163	132	108	89	75	63	53	45	33	30	30	30	30	30
1	166	135	111	92	77	65	55	47	35	30	30	30	30	30
2	0	156	130	110	94	81	69	60	45	34	30	30	30	30
3	0	169	143	122	105	91	79	69	53	40	31	30	30	30
4	0	178	152	131	113	99	86	75	58	45	36	30	30	30
5	0	0	159	138	120	105	92	81	63	49	39	31	30	30
6	0	0	166	144	126	110	97	86	67	53	42	33	30	30
7	0	0	171	149	130	115	101	90	71	56	45	36	30	30
8	0	0	175	153	135	119	105	93	74	59	47	38	30	30
9	0	0	180	157	139	123	109	97	77	62	49	40	32	30
10	0	0	0	161	142	126	112	100	80	64	52	42	34	30
20	0	0	0	0	166	149	134	121	99	81	67	55	46	38
25	0	0	0	0	174	157	142	129	106	88	73	60	50	42
30	0	0	0	0	0	164	148	135	112	93	77	65	54	45
40	0	0	0	0	0	174	159	145	121	102	85	72	61	51
50	0	0	0	0	0	0	167	153	129	109	92	78	66	56
60	0	0	0	0	0	0	174	160	135	115	97	83	71	60
70	0	0	0	0	0	0	180	165	140	120	102	87	75	64
80	0	0	0	0	0	0	0	170	145	124	107	91	79	68
90	0	0	0	0	0	0	0	175	150	128	110	95	82	71
100	0	0	0	0	0	0	0	179	153	132	114	98	85	74

Avoid placing thicknesses > 180 mm and < 30 mm

ASPHALT CONCRETE TYPE I

OVERLAY DESIGN TABLE NO. 1 | **E = 4000MPa**

Traffic x10^6	Equivalent modulus in MPa - Overlay thickness in mm													
	25	50	75	100	125	150	175	200	250	300	350	400	450	500
.3	106	85	70	58	49	42	36	31	30	30	30	30	30	30
.4	112	91	75	63	55	45	39	34	30	30	30	30	30	30
.5	117	95	79	66	56	48	42	36	30	30	30	30	30	30
.6	121	99	82	70	59	51	44	38	30	30	30	30	30	30
.7	124	102	86	72	62	53	46	40	31	30	30	30	30	30
.8	127	105	88	75	64	55	48	42	32	30	30	30	30	30
.9	130	108	91	77	66	57	50	43	34	30	30	30	30	30
1	132	110	93	79	68	59	51	45	35	30	30	30	30	30
2	150	128	109	94	82	72	63	56	44	35	30	30	30	30
3	161	138	120	104	91	80	71	63	50	40	33	30	30	30
4	169	146	127	112	98	87	77	69	55	45	37	30	30	30
5	179	153	134	118	104	92	82	74	59	48	40	33	30	30
6	-	158	139	123	109	97	87	78	63	52	42	35	30	30
7	-	163	143	127	113	101	90	81	66	54	45	37	31	30
8	-	167	147	131	117	104	94	85	69	57	47	39	33	30
9	-	171	151	132	120	108	97	87	72	59	49	41	34	30
10	-	174	154	137	123	111	100	90	74	61	51	43	36	30
20	-	-	177	159	144	131	119	109	91	77	65	55	47	40
25	-	-	-	167	151	138	126	115	97	82	70	60	51	44
30	-	-	-	173	157	144	132	121	102	87	74	64	55	47
40	-	-	-	-	167	153	141	130	111	95	82	70	61	51
50	-	-	-	-	175	161	148	137	118	101	87	76	66	57
60	-	-	-	-	-	167	155	143	123	107	92	80	70	61
70	-	-	-	-	-	173	160	148	128	111	97	85	74	65
80	-	-	-	-	-	177	166	153	133	115	101	88	77	68
90	-	-	-	-	-	-	169	157	137	119	104	92	80	71

Avoid placing thicknesses > 180 mm and < 30 mm

DENSE BITUMINOUS MACADAM

OVERLAY DESIGN TABLE NO. 2 | **E = 5000 MPa**

Traffic x10^6	Equivalent modulus in MPa - Overlay thickness in mm													
	25	50	75	100	125	150	175	200	250	300	350	400	450	500
.3	111	93	78	75	75	75	75	75	75	75	75	75	75	75
.4	118	100	85	75	75	75	75	75	75	75	75	75	75	75
.5	124	106	91	79	75	75	75	75	75	75	75	75	75	75
.6	129	111	95	83	75	75	75	75	75	75	75	75	75	75
.7	134	115	100	87	76	75	75	75	75	75	75	75	75	75
.8	138	119	103	90	80	75	75	75	75	75	75	75	75	75
.9	141	123	107	94	83	75	75	75	75	75	75	75	75	75
1	144	126	110	97	86	76	75	75	75	75	75	75	75	75
2	167	149	132	118	106	95	86	78	75	75	75.	75	75	75
3	181	164	147	132	119	108	98	90	75	75	75	75	75	75
4	192	174	157	143	129	118	108	99	83	75	75	75	75	75
5	200	183	166	151	138	126	116	106	90	77	75	75	75	75
6	207	190	173	158	145	133	122	113	96	83	75	75	75	75
7	213	197	180	164	151	139	128	118	101	87	76	75	75	75
8	219	202	185	170	156	144	133	123	106	92	80	75	75	75
9	223	207	190	175	161	149	138	128	110	96	83	75	75	75
10	227	211	194	179	165	153	142	132	114	99	87	76	75	75
20	0	241	224	209	195	183	171	160	141	125	111	99	89	79
25	0	0	234	219	205	193	181	170	151	135	120	108	96	87
30	0	0	243	227	214	201	189	178	159	142	128	115	103	93
40	0	0	0	241	227	214	202	191	172	155	140	126	114	104
50	0	0	0	0	237	224	213	202	182	165	149	136	124	113
60	0	0	0	0	246	233	221	210	191	173	158	144	131	120
70	0	0	0	0	0	240	229	218	198	180	165	151	138	126
80	0	0	0	0	0	247	235	224	204	187	171	157	144	132
90	0	0	0	0	0	0	241	230	210	192	176	162	149	137
100	0	0	0	0	0	0	246	235	215	197	181	167	154	142
50mm AC Surfacing														
10	182	166	149	134	120	108	97	87	75	75	75	75	75	75
20	211	196	179	164	150	138	126	115	96	80	75	75	75	75
25	220	206	189	174	160	148	136	125	106	89	75	75	75	75
30	228	214	198	182	169	156	144	133	114	97	83	75	75	75
40	240	226	210	196	182	169	157	146	127	110	95	81	75	75
50	249	236	221	206	192	179	168	157	137	120	104	91	79	75
60	0	244	229	214	202	188	176	165	146	128	113	99	86	75
75mm AC Surfacing														
60	231	219	204	189	176	163	151	140	121	103	88	75	75	75
70	238	226	211	196	183	170	159	148	128	110	95	81	75	75
80	243	231	217	202	189	177	165	154	134	117	101	87	75	75
90	248	237	222	208	195	182	171	160	140	122	106	92	79	75
100	0	241	227	213	199	187	176	165	145	127	111	97	84	75

Avoid placing thicknesses > 250 mm and < 75 mm

BITUMEN STABILISED MATERIAL (BSM175)

OVERLAY DESIGN TABLE NO. 3 | E = 2000 MPa

Traffic x10^6	Equivalent modulus in MPa - Overlay thickness in mm								
	40	50	60	80	100	125	150	175	200
.3	125	113	102	84	75	75	75	75	75
.4	132	120	108	90	75	75	75	75	75
.5	138	125	114	95	80	75	75	75	75
.6	142	129	118	99	83	75	75	75	75
.7	146	133	122	102	86	75	75	75	75
.8	150	137	125	105	89	75	75	75	75
.9	153	140	128	108	92	76	75	75	75
1	156	142	131	110	94	78	75	75	75
2	175	161	149	128	110	93	78	75	75
3	187	173	160	139	121	102	87	75	75
4	195	181	169	147	128	109	94	80	75
5	202	188	175	153	134	115	99	85	75
6	207	193	180	158	140	120	103	89	78
7	212	198	188	163	144	124	107	93	81
8	216	202	189	167	148	128	111	96	84
9	219	206	193	170	151	131	114	99	87
10	223	209	196	174	154	134	117	102	89
20	244	230	218	185	175	154	136	120	107
25	0	237	225	202	182	161	143	127	113
30	0	243	231	208	188	167	148	132	118
40	0	0	240	217	197	176	157	141	126
50	0	0	247	224	205	183	164	147	133
60	0	0	0	230	211	189	170	153	138
50mm AC Surfacing									
10	178	164	151	129	109	89	75	75	75
20	199	185	173	150	130	109	91	75	75
25	206	192	180	157	137	116	98	82	75
75mm AC Surfacing									
25	181	167	155	132	112	91	75	75	75
30	187	173	161	138	118	97	78	75	75
40	196	182	170	147	127	106	87	75	75
50	203	189	177	154	135	113	94	77	75
60	209	195	183	160	141	119	100	83	75

Avoid placing thicknesses > 250 mm and < 75 mm

DENSE BITUMINOUS MACADAM

OVERLAY DESIGN TABLE NO. 2 | E = 5000 MPa

Traffic x10^6	Equivalent modulus in MPa - Overlay thickness in mm													
	25	50	75	100	125	150	175	200	250	300	350	400	450	500
.3	228	210	198	190	183	177	172	167	160	153	150	150	150	150
.4	232	213	201	193	186	108	175	170	163	156	150	150	150	150
.5	235	216	204	195	188	182	177	173	165	158	152	150	150	150
.6	237	218	206	198	190	184	179	175	167	160	154	150	150	150
.7	239	220	208	199	192	196	181	176	168	162	156	151	150	150
.8	241	222	210	201	194	198	183	178	170	163	157	152	150	150
.9	243	224	211	203	195	199	184	179	171	164	159	153	150	150
1	245	225	213	204	197	191	185	181	172	166	160	155	150	150
2	256	235	223	214	206	200	194	189	181	174	168	162	158	153
3	263	242	229	220	212	206	200	195	186	179	173	168	163	158
4	268	247	234	224	217	210	204	199	191	183	177	172	167	162
5	273	251	238	228	220	214	208	203	194	187	180	175	170	165
6	277	255	241	231	224	217	211	206	197	190	183	177	172	168
7	280	258	244	234	226	220	214	208	200	192	186	180	175	170
8	283	260	247	237	229	222	216	211	202	194	188	182	177	172
9	285	263	249	239	231	224	218	213	204	196	190	184	179	174
10	288	265	251	241	2333	226	220	215	206	198	191	186	180	176
20	0	281	267	256	248	241	234	229	219	211	204	198	193	188
25	0	287	273	262	253	246	239	234	224	216	209	203	197	193
30	0	292	277	266	258	250	244	238	228	220	213	207	201	196
40	0	0	285	274	265	258	251	245	235	227	220	213	208	203
50	0	0	292	281	271	264	257	251	241	233	225	219	213	208
60	0	0	298	286	277	269	262	256	246	238	230	224	218	213
70	0	0	0	291	282	274	267	261	251	242	234	228	222	217
80	0	0	0	295	286	278	271	264	255	246	238	232	226	220
90	0	0	0	299	290	282	275	269	258	249	242	235	229	224
100	0	0	0	0	294	286	279	272	262	253	245	238	232	227
50mm AC Surfacing														
10	243	220	206	196	188	181	175	170	161	153	150	150	150	150
20	260	236	222	211	203	196	189	184	174	166	159	153	150	150
25	266	242	228	217	208	201	194	189	179	171	164	158	152	150
75mm AC Surfacing														
25	241	217	203	192	183	176	169	164	154	150	150	150	150	150
30	247	222	207	196	188	180	174	168	158	150	150	150	150	150
40	256	231	215	204	195	188	181	175	165	157	150	150	150	150
50	263	237	222	211	201	194	187	181	171	163	155	150	150	150
60	269	243	228	216	207	199	192	186	176	168	160	154	150	150
70	275	249	233	221	212	204	197	191	181	172	164	158	152	150
80	280	253	237	225	216	208	201	195	185	176	168	162	156	150
90	285	258	241	229	220	212	205	199	188	179	172	165	159	154
100	289	262	245	233	224	216	209	202	192	183	175	168	162	157

Avoid placing thicknesses > 300 mm and < 150 mm

HYDRAULICALLY BOUND STONE (HBS6)

OVERLAY DESIGN TABLE NO. 5 | **E = 7000 MPa**

Traffic x10^6	**Equivalent modulus in MPa -** Overlay thickness in mm													
	25	50	75	100	125	150	175	200	250	300	350	400	450	500
2	31	28	26	249	237	227	218	210	198	188	180	172	166	161
3	32	29	27	256	244	234	225	217	205	194	186	179	173	167
4	33	29	27	262	249	239	230	222	210	199	191	183	177	172
5	33	30	28	266	254	243	234	226	214	203	195	187	181	175
6	34	30	28	270	257	247	238	230	217	207	198	190	184	179
7	34	31	28	273	260	250	241	233	220	209	201	193	187	181
8	34	31	29	276	263	253	244	236	223	212	203	196	189	184
9	35	31	29	279	266	255	246	238	225	214	205	198	192	186
10	35	31	29	281	268	257	248	240	227	216	208	200	194	188
75mm AC Surfacing														
10	329	294	272	256	243	232	223	215	202	191	183	175	169	163
20	348	312	289	273	259	248	239	231	217	206	197	190	183	177
25	355	318	295	279	265	254	245	236	223	212	202	195	188	182
100mm AC Surfacing														
25	330	293	270	254	240	220	220	211	198	187	177	170	163	157
30	335	298	275	258	245	224	224	216	202	191	182	174	167	161
40	345	307	284	267	253	232	232	224	210	199	189	181	174	168
50	352	314	291	273	260	239	239	230	216	205	195	187	180	174
60	359	320	297	279	265	244	244	236	221	210	200	192	185	179

Avoid placing thicknesses > 400 mm and < 125 mm

HYDRAULICALLY BOUND STONE (HBS3)

OVERLAY DESIGN TABLE NO. 6 | **E = 4000 MPa**

Traffic x10^6	Equivalent modulus in MPa - Overlay thickness in mm									
	400	500	600	800	1000	1250	1500	2000	2500	3000
.3	252	220	193	149	114	100	100	100	100	100
.4	266	234	207	163	128	100	100	100	100	100
.5	277	245	218	175	139	103	100	100	100	100
.6	286	254	228	184	148	112	100	100	100	100
.7	294	262	235	192	156	120	100	100	100	100
.8	301	269	242	198	163	127	100	100	100	100
.9	307	275	248	204	169	133	102	100	100	100
1	312	280	254	210	175	138	107	100	100	100
2	349	317	290	246	211	174	144	100	100	100
3	0	339	312	268	233	196	165	115	100	100
4	0	0	327	284	248	212	181	131	100	100
5	0	0	340	296	261	224	193	143	103	100
6	0	0	350	306	271	234	203	153	113	100
7	0	0	0	315	279	243	212	162	121	100
8	0	0	0	322	287	250	220	169	129	100
9	0	0	0	329	293	257	226	176	135	101
10	0	0	0	335	299	263	232	182	141	107
50mm AC Surfacing										
3	325	294	267	223	188	151	120	100	100	100
4	341	309	282	239	203	167	136	100	100	100
5	0	321	295	251	216	179	148	100	100	100
6	0	332	305	261	226	189	158	108	100	100
7	0	340	313	270	234	198	167	117	100	100
8	0	348	321	277	242	205	175	124	100	100
9	0	0	328	284	248	212	181	131	100	100
10	0	0	333	290	254	218	187	137	100	100

Avoid placing thicknesses > 350 mm and < 100 mm

1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 Appendices

GRADED CRUSHED STONE (GCS-A)

OVERLAY DESIGN TABLE NO. 7 | E = 400 MPa

Traffic x10^6	Equivalent modulus in MPa - Overlay thickness in mm										
	25	50	75	100	125	150	175	200	250	300	350
.3	272	241	218	165	150	128	114	101	100	100	100
.4	288	255	231	175	159	136	120	107	100	100	100
.5	301	267	242	184	166	142	126	112	101	100	100
.6	313	277	251	191	173	148	131	116	105	100	100
.7	323	286	259	197	178	152	135	120	108	100	100
.8	332	294	266	202	183	157	139	123	111	100	100
.9	347	301	273	207	188	161	142	126	114	100	100
1	400	308	279	212	192	164	145	129	117	100	100
2	0	355	321	244	221	189	168	148	134	115	100
3	0	385	308	265	240	205	182	161	146	125	100
4	0	0	349	281	254	218	193	171	155	133	106
5	0	0	370	294	266	228	202	179	162	139	111
6	0	0	388	305	276	237	210	186	168	144	116
7	0	0	0	315	285	244	216	192	174	149	119
8	0	0	0	324	293	251	222	197	179	153	123
9	0	0	0	331	300	257	228	202	183	157	126
10	0	0	0	339	307	263	233	206	187	160	128

Avoid placing thicknesses > 400 mm and < 100 mm

Appendix B:

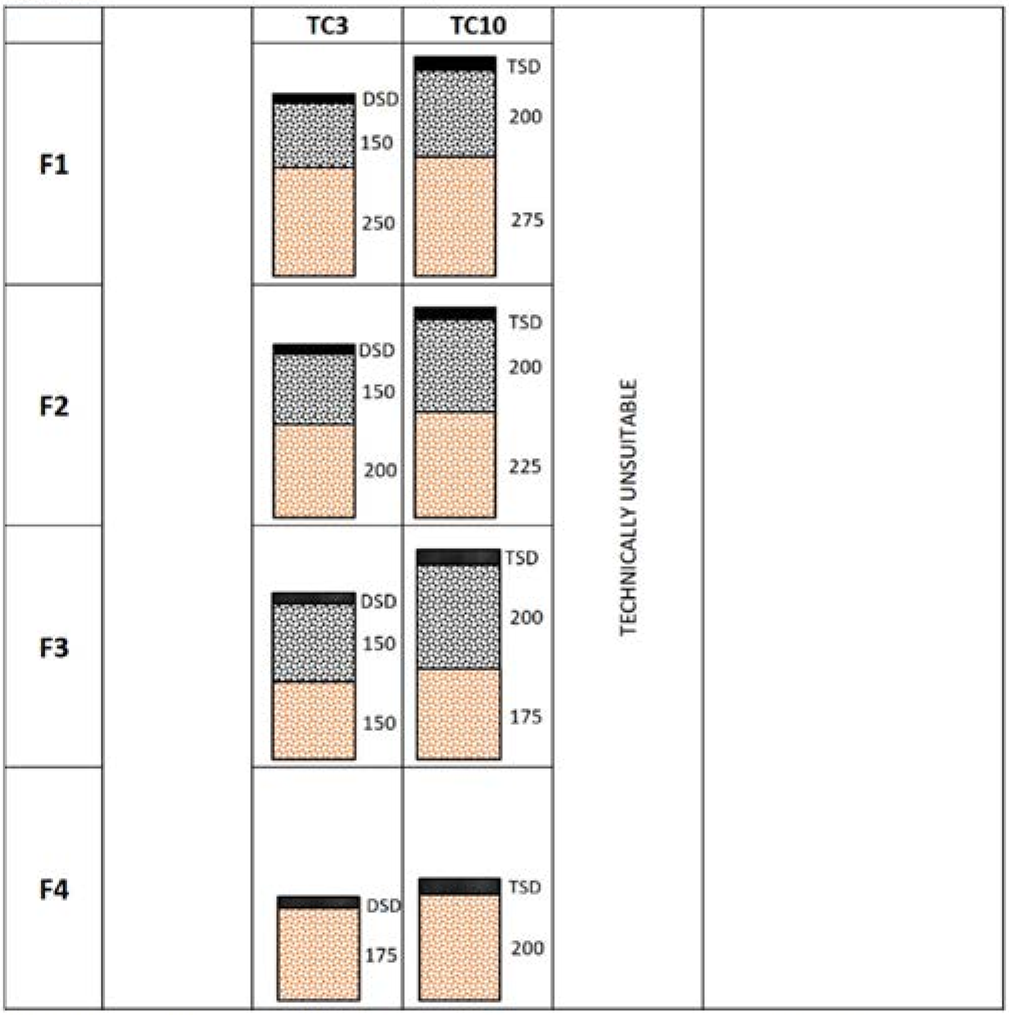
Example of Structural Number Table (RDM 3.4)

MINISTRY OF ROADS AND TRANSPORT

STANDARD PAVEMENT STRUCTURES FOR MEDIUM, HEAVY AND VERY HEAVY VOLUME ROADS

STANDARD PAVEMENT STRUCTURE TYPE 1

	SURFACING:	Double or Triple Surface Dressing
	BASE:	Natural Gravel(G80)
	SUB-BASE:	Natural Gravel (G30) or Hand Packed Stone



TRAFFIC CLASSIFICATION

Design Traffic Class	Design Traffic Range (M CESA)
TC3	1-3
TC10	3-10
TC17	10-17
TC30	17-30
TC50	30-50
TC80	50-80
TC150	80-150

SUBGRADE CLASSIFICATION

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

FOUNDATION CLASSIFICATION

Foundation Class	Effective Surface Modulus (MPa)	Equivalent Subgrade Class
F1	75	S3
F2	95	S4
F3	130	S5
F4	200	S6
F5	400	-

IMPROVED SUBGRADE

Native Subgrade	S1			S2			S3			S4	
Capping Material	G8	G10	G14	G10	G14	G23	G14	G23	G45	G23	G45
Thickness (mm)	375	400	425	150	150	175	150	150	200	150	150
New Subgrade	S2	S3	S4	S3	S4	S4	S4	S5	S5	S5	S6
Foundation Class	-	F1	F2	F1	F1	F2	F2	F3	F4	F3	F4

*Other capping options available in RDM 3.3. DSD = Double Surface Dressing, TSD = Triple Surface Dressing

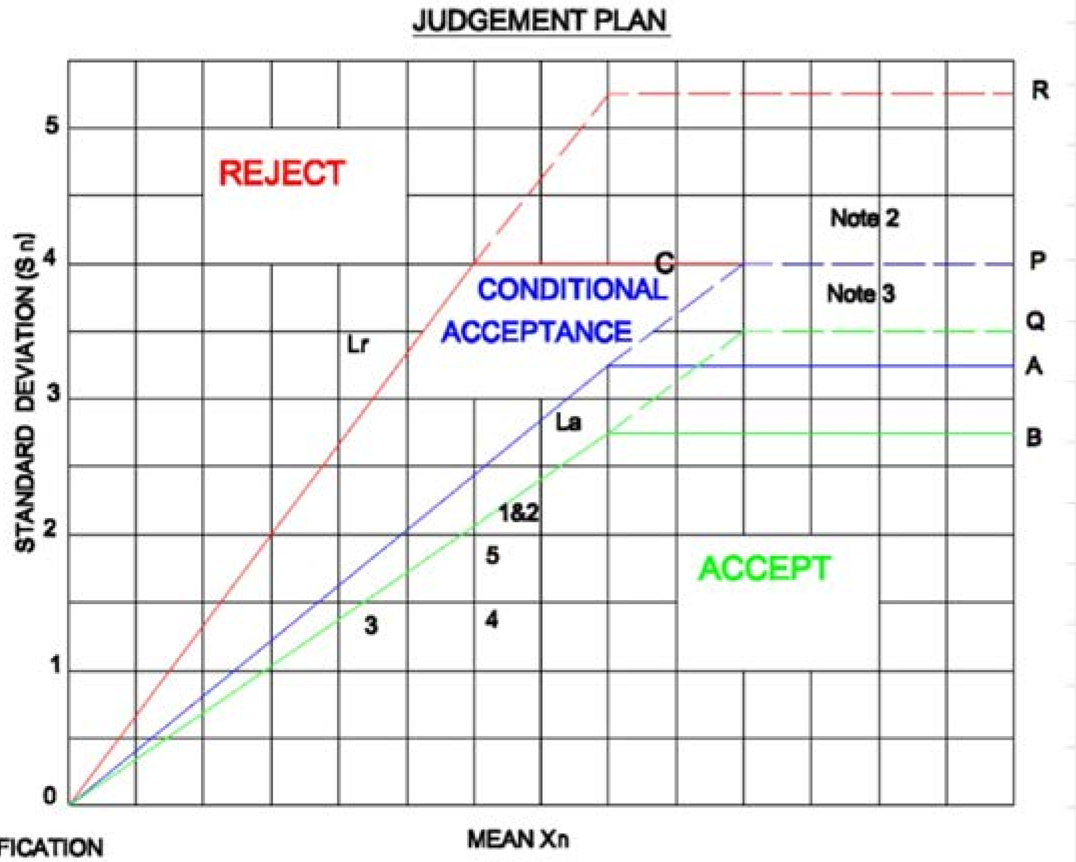
Appendix B:

Target SN_d for Standard Pavement Type 1

Foundation Foundation	Layer	Type	Layer Strength Coefficient	Thicknesses (mm) TC3	TC10	TC17	TC30	TC50	TC80	TC150	Structural Numbers (SN) TC3	TC10	TC17	TC30	TC50	TC80	TC150
F1	Surfacing	ST	0	DSD	TSD												
	Base	Natural G80	0.12	150	200						0.65	0.87					
	Subbase	Natural G30	0.105	250	275						1.03	1.14					
	Target SN										1.68	2.00					
F2	Surfacing	ST	0	DSD	TSD												
	Base	Natural G80	0.11	150	200						0.65	0.87					
	Subbase	Natural G30	0.105	200	225						0.83	0.93					
	Target SN										1.48	1.80					
F3	Surfacing	ST	0	DSD	TSD												
	Base	Natural G80	0.11	150	200						0.65	0.87					
	Subbase	Natural G30	0.105	150	175						0.62	0.72					
	Target SN										1.27	1.59					
F4	Surfacing	ST	0	DSD	TSD												
	Base	Natural G80	0.11	175	200						0.76	0.87					
	Subbase	Natural G30	0.105								0.00	0.00					
	Target SN										0.76	0.87					

Appendix C:

Compaction Judgement Chart



SPECIFICATION LIMIT[1] Ls

	87	88	89	90	91	92	93	94
2&3	89	90	91	92	93	94	95	96
1	91	92	93	94	95	96	97	98
5	93	94	95	96	97	98	99	100
4	95	96	97	98	99	100	101	102
	97	98	99	100	101	102	103	104
	99	100	101	102	103	104	105	106

VARIABLE SCALE FOR RELEVANT L_s

A = Initial submission limit $L_s \geq 93$ P = Initial submission limit $L_s \leq 91$

B = Resubmission limit $L_s \geq 93$ Q = Resubmission limit $L_s \leq 91$

C = Rejection limit $L_s \geq 93$ R = Rejction limit $L_s \leq 91$

NOTE (1) Circle relevant L_s and put reference number of lot next to L_s

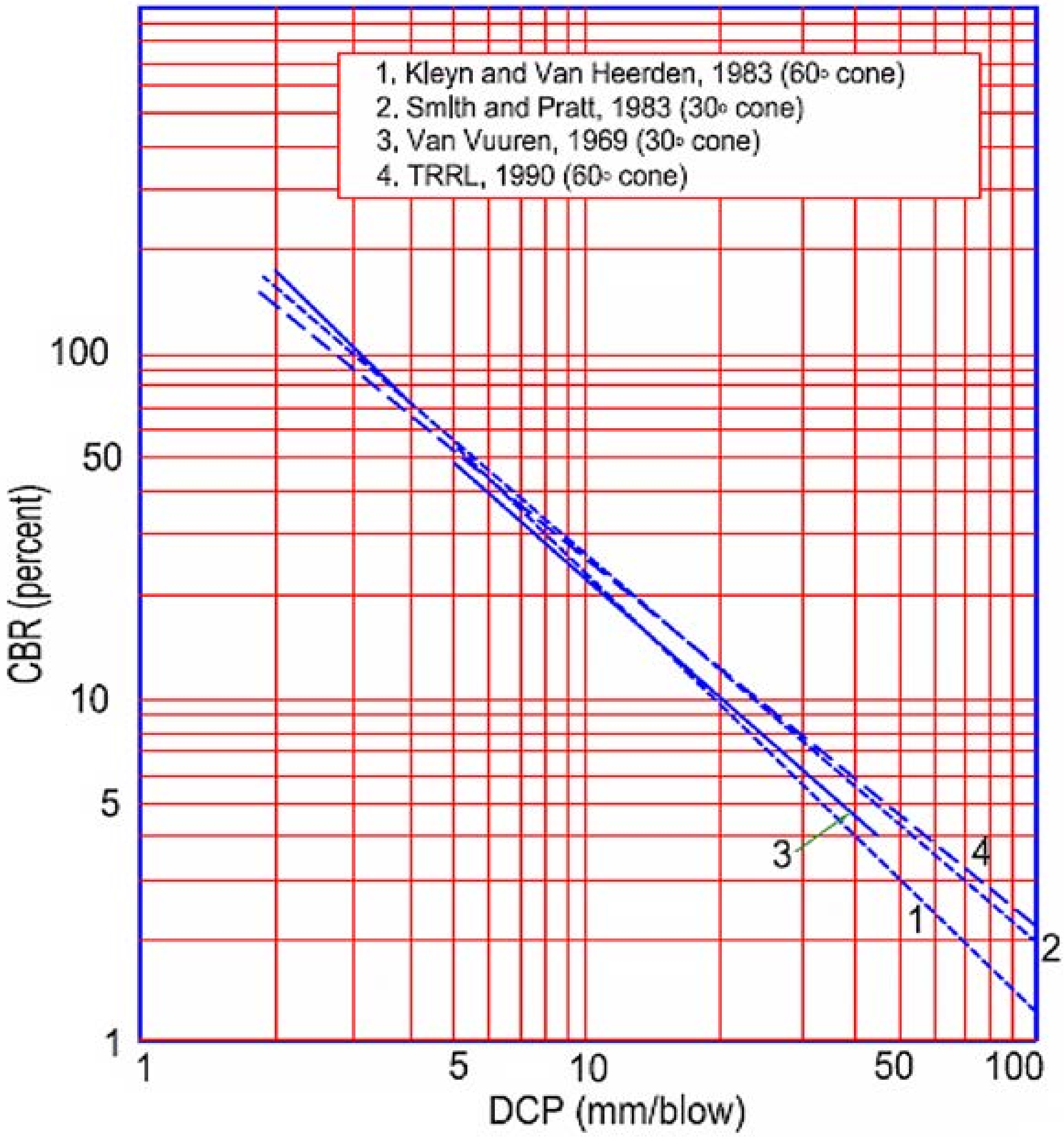
(2) Reject for $L_s \geq 93$ Conditional for $L_s \leq 91$

(3) Conditional for $L_s \geq 93$ Accept for $L_s \leq 91$

Appendix D:

Calculation of CBR and Subgrade E-Modulus from DCP

The calculations of the CBRs are given in the graphs and equations below.



1. $\text{Log}_{10}(\text{CBR}) = 2{,}632 - 1{,}28\text{Log}_{10}(\text{mm/blow})$

2. $\text{Log}_{10}(\text{CBR}) = 2{,}555 - 1{,}145\text{Log}_{10}(\text{mm/blow})$

3. $\text{Log}_{10}(\text{CBR}) = 2.503 - 1.15\text{Log}_{10}(\text{mm/blow})$

4. $\text{Log}_{10}(\text{CBR}) = 2{,}48 - 1{,}057\text{Log}_{10}(\text{mm/blow})$

The E-modulus of subgrade can also be calculated from DCP penetration using the equation below:

$$E_{SG} = 357.87\ x\text{DCP}^{0.6445}$$

This applies for range: lower limit = 5 x CBR and upper limit = 20 x CBR

Appendix E:

Worked Example – Pavement Evaluation and Overlay Design

This is an example showing the different processes, stages and illustration of results in the evaluation of the existing pavement and overlay design. The example is derived from a real project involving Pavement Evaluation and Overlay Design for Garsen-Witu-Lamu Road carried out by the Materials Testing and Research Division (MTRD).

Road length: 86.8 km.

Visual condition surveys were conducted, and the road was divided into 4 uniform sections based on surface condition. The main defects were cracking and deformation.

Section 1: Km 0+000 to Km 45+200 – 45.2 Km

Section 2: Km 49+500 to Km 63+305 – 13.8 Km

Section 3: Km 84+900 to Km 88+400 – 3.5 Km

Section 4: Km 89+150 to Km 113+500 – 24.3 Km

Longitudinal and crocodile cracks on wheel paths, transverse cracks also on wheel paths. Crack observed on newly sealed sections.

Table E1 Visual Condition Survey

Section	Condition
1	Deformation and depressions, and water ponding on the road and some longitudinal cracks
2	No cracks on newly sealed sections except localised longitudinal cracks in 1 position.
3	No notable defects
4	No cracks were observed on newly sealed sections, but observed on previously resealed sections. Bleeding was also observed.

This is typical of where reseals conceal defects such as cracking and the surveyor might have the wrong impression. This is where historical data such as maintenance history becomes paramount.

Roughness surveys were carried out and the criteria used is given in Table E2:

Table E2 Road Roughness Rating Standards for IRI

Roughness, m/km	Rating Description
0-2	Very Good
2-4	Good
4-6	Fair
6-10	Poor
Above 10	Bad

Table E3 Roughness Measurements and Rating of IRI of the Road

Lane	Min IRI m/Km	Max IRI m/Km	Mean IRI m/Km	Mean IRI value rating	Standard deviation IRI (m/Km)
LHS	0.7	5.5	1.6	Very Good	0.4
RHS	0.2	5.8	1.6	Very Good	0.4
Mean	0.5	5.7	1.6	Very Good	0.4

Table E4 Present Serviceability Rating (PSR)

PSR	Rating Description
0-1	Bad
1-2	Poor
2-3	Fair
3-4	Good
4-5	Very Good

Table E5 Measured Values

HS	Length (km)	Point Score							Point Summary				PSR
		A	B	C	D	E	F	G	Σ A-G Max: 40	Mean	%	Rating	
1	45.2	4.0	5.0	4.0	5.0	4.0	5.0	0.0	27.0	3.9	67.5	Good	3.9
2	13.8	4.0	4.0	4.0	5.0	5.0	5.0	0.0	27.0	3.9	67.5	Good	3.9
3	3.5	4.0	4.0	4.0	5.0	5.0	5.0	5.0	32.0	4.6	80.0	V. Good	4.6
4	24.1	4.0	4.0	4.0	5.0	5.0	5.0	0.0	27.0	3.9	67.5	Good	3.9
∑	**87**	**16**	**17**	**16**	**20**	**19**	**20**	**5**	**113**	**16.1**	**215.0**		**16.1**
Mean		**4.0**	**4.3**	**4.0**	**5.0**	**4.8**	**5.0**	**1.3**	**28.3**	**4.0**	**71.7**	**Good**	**4.0**

Note:
A: General Appearance, B: Surface Texture, C: Bitumen Condition, D: Potholes, E: Surface Irregularity, F: Rutting, G: Cracking

Conclusion

Surface condition is generally good and both criteria have not exceeded threshold values to trigger intervention. However, there is possible reflective cracking occurring in the new surfacing and this is detrimental.

The next stage is to determine the thicknesses of the existing pavement layers as shown in Table E6.

Tests results required from the cores are illustrated in Table E7. Some of the cores are used for the Marshall tests and results are shown in Table E8.

Table E6 Trenching Thicknesses Summary

Section	Chainage	Layer	Layer Thickness (mm)	Section Mean Layer Thickness (mm)
1	2+215	Base	150	157
1	23+750	Base	170	
1	42+120	Base	150	
2	51+150	Base	150	150
2	62+590	Base	150	
4	102+510	Base	150	150
4	113+210	Base	150	
4	96+600	Base	150	
1	2+215	Subbase	200	180
1	23+750	Subbase	190	
1	42+120	Subbase	150	
2	51+150	Subbase	180	190
2	62+590	Subbase	200	
4	102+510	Subbase	200	193
4	113+210	Subbase	200	
4	96+600	Subbase	180	
1	2+215	Subgrade	350	303
1	23+750	Subgrade	290	
1	42+120	Subgrade	270	
2	51+150	Subgrade	310	295
2	62+590	Subgrade	280	
4	102+510	Subgrade	270	287
4	113+210	Subgrade	290	
4	96+600	Subgrade	300	

1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
Appendices

Table E7 Summary of AC Core Results

Chain-age	Core Thickness mm	Binder Content %	Core Density	Core Voids	PRD	Refusal voids	VMA	VFB	Remark on the core	Type of crack
2+215	53	5.3	**2.299**	7	95.6	2.7	15.1	82	Good	None
6+022	53	5.3	**2.309**	6.6	95	1.7	14.2	88.2	Good	None
12+040	62	5.1	**2.287**	7.7	96.3	4.1	15.9	74	Good	None
23+768	54	5.2	**2.222**	10.2	94.2	4.6	16.6	72	Good	None
28+810	60	5.2	**2.231**	9.9	94.9	5	16.9	70.3	Good	None
33+250	54	5.3	**2.286**	7.5	97.4	5	17.1	70.8	Cracked	LC
33+250	52	5.3	**2.268**	8.2	95.9	4.3	16.5	73.7	Cracked	LC
42+120	53	5.2	**2.335**	5.7	96.6	2.4	14.6	83.7	Cracked	LC
51+150	46	5.3	**2.166**	9.9	94	4.1	15.9	74.5	Cracked	TC
51+150	46	5.3	**2.19**	8.9	93	1.9	14.1	86.2	Cracked	LC
56+070	44	5.2	**2.251**	6.4	95.3	1.8	13.8	86.7	Cracked	None
62+590	56	5.1	**2.274**	5.6	97.4	3.1	14.7	78.6	Good	None
89+630	58	5.2	**2.233**	7.2	95.8	3.1	14.9	79.2	Good	None
96+600	57	4.9	**2.207**	8.7	96.2	5	15.9	68.5	Cracked	TC
96+601	59	5.2	**2.176**	9.6	94.6	4.4	16	72.5	Cracked	LC
102+510	36	5.3	**2.202**	8.4	94.2	2.7	14.8	81.5	Cracked	LC
107+000	58	5.1	**2.304**	4.4	100.3	4.7	16	70.9	Good	None
113+200	61	5.2	**2.224**	7.6	96.9	4.6	16.2	71.7	Cracked	None
					>95	**>3.0**				

Table E8 AC Mixes Summary

Sampling Point	Marshall Test and Binder Extraction					
	Binder Content	Marshall Density	Max. Sp. Gr.	Marshall Voids	Marshall Stability	Marshall Flow
84+763 LHS	5.3	2.213	**2.403**	7.9	14.6	3.2
84+865 RHS	5.2	2.213	**2.406**	8.0	13.2	3.0
	5.3	2.213	**2.404**	8.0	13.9	3.1
Requirements						

During coring samples of bound bases are obtained and subjected to moisture and strength tests shown in Table E9.

Table E9 Summary of Base UCS Analysis

CP	Chainage	Core Thickness mm	Bulk Density Kg/m³	Moisture Content	Dry Density	UCS KN/ m²	Surfacing Remarks	Remark
1	Km 2+215	122	1781	1.7	1752	558	Good	Good
2	Km 6+022	134	1900	3.1	1843	954	Good	Good
3	Km 12+040	117	1882	4.1	1808	1908	Good	Good
4	Km 23+768	159	1938	4.1	1862	1402	Good	Good
5	Km 28+810	112	2079	4.8	1983	6316	Good	Good
6	Km 33+250	126	2098	4.6	2006	5787	Cracked	Cracked
7	Km 33+250	106	2090	5.6	1980	8148	Cracked	Good
8	Km 42+120	112	1960	3.4	1895	1349	Cracked	Cracked
9	Km 51+150	147	2108	4.7	2014	2710	Cracked	Cracked
10	Km 51+150	148	2045	3.5	1976	268	Cracked	Cracked
11	Km 56+070	134	2066	3.4	1998	1838	Cracked	Good
12	Km 62+590	158	1968	4.2	1889	2786	Good	Good
13	Km 89+630	97	2108	3	2047	4123	Good	Good
14	Km 96+600	143	2044	4.6	1955	2204	Cracked	Cracked
15	Km 96+601	116	2007	5.6	1901	5362	Cracked	Cracked
16	Km 102+510	144	2061	4.4	1974	2355	Cracked	Cracked
17	Km 107+000	85	1856	3.8	1788	518	Good	Good
	Average	**127.1**	**1999.5**	**4.0**	**1921.7**	**2857.9**		

For base materials, samples are collected and tested in the laboratory for in-situ densities (Table E10) as well as field moistures. Additionally, tests are carried out to determine index properties (Table E11).

Table E10 Test Results for Field Densities for Base Layer

CP	Chainage	FDD Kg/m³	PMC %	MDD - Kg/m³	OMC %	Relative Compaction %	Relative Moisture Content %
1	2+215	1792	10.5	1820	10	98	105
2	23+750	1840	10.5	1796	12.6	103	83
3	42+120	1757	9.8	1822	9.5	96	103
4	51+150	1939	9.8	1866	10.6	104	92
5	62+590	1932	8.1	1939	8.3	100	99
6	96+600	1901	9.8	1889	10.1	101	97
7	102+510	1882	9.2	1862	10.4	101	88
8	113+210	1980	7.8	1906	8.3	104	94

1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
Appendices

Table E11 Summary of Index properties for Base Layer

TP	Chainage	Offset	LL	PL	PI	LS	PM	MDD	OMC	NMC	CBR	Swell
1	2+215	LHS	NP	NP	NP	NP	0	1820	10	9.8	70	0.1
2	23+750	RHS	30	20	10	6	340	1796	12.6	10.5	70	0.1
3	42+120	LHS	30	20	10	5	340	1822	9.5	9.2	60	0.1
4	51+150	RHS	31	23	8	4	240	1866	10.6	9.6	90	0.1
5	62+590	RHS	26	19	7	4	378	1939	8.3	7.8	100	0.1
6	96+600	LHS	NP	NP	NP	NP	0	1889	10.1	9.2	90	0.1
7	102+510	RHS	NP	NP	NP	NP	0	1862	10.4	11.5	100	0.1
8	113+210	LHS	NP	NP	NP	NP	0	1906	8.3	7.9	100	0.1

Similarly, subbases are tested for in-situ properties including density, pavement moisture and UCS from core (where in this case the subbase is bound) as well index properties in sections where SB is unbound. Results are illustrated in Table E12, Table E13 and Table E14.

Table E12 Summary of Subbase UCS Analysis

CP	Chainage	Core Thickness mm	Bulk Density Kg/m^3	Moisture Content	Dry Density	UCS KN/m^2	Surfacing Remarks	Base Remarks
1	Km 2+215	157	1778	3.8	1713	384	Good	
2	Km 6+022	120	1804	2.5	1760	721	Good	
3	Km 23+768	135	1894	3.3	1833	355	Good	
4	Km 51+150	128	2066	2.8	2010	1623	Cracked	Cracked
5	Km 102+510	106	2023	3.6	1953	1048	Cracked	Cracked
	AVERAGE	129	1913	3	1854	826		

Table E13 Test Results for Field Densities for Subbase Layer

TP	Chainage	Offsett	Layer Thickness mm	FDD Kg/m^3	PMC %	MDD Kg/m^3	OMC %	Relative Compaction	Relative Moisture Content %
1	2+215	LHS	150	1795	10.2	1808	11.3	99	90
2	23+750	RHS	160	1770	9.8	1756	11.6	101	84
3	42+120	RHS	150	1763	9.9	1893	9	93	110
4	51+150	RHS	180	1813	9.2	1908	10.6	95	87
5	62+590	RHS	200	1779	11.4	1888	10.5	94	109
6	96+600	LHS	180	1820	10.8	1744	12.8	104	84
7	102+510	RHS	200	2000	7.8	1902	8.5	105	92
8	113+210	LHS	180	1930	9.2	1822	10.7	106	86

Table E14 Summary of Index properties for Subbase Layer

TP	Chainage	Offsett	LL	PL	PI	LS	PM	MDD	OMC	NMC	CBR	Swell
1	2+215	LHS	NON - PLASTIC				0	1808	11.3	10.3	60	0.1
2	23+750	RHS	NON - PLASTIC				0	1756	11.7	9.9	50	0.1
3	42+120	LHS	33	18	15	8	735	1893	9	11.5	40	0.1
4	51+150	RHS	32	18	14	7	602	1908	10	9.3	50	0.1
5	62+590	RHS	27	18	9	5	450	1888	10.5	8.3	50	0.1
6	96+600	LHS	35	22	13	6	195	1744	12.8	11.4	60	0.1
7	102+510	RHS	34	21	13	6	624	1902	8.5	7.3	70	0.1
8	113+210	LHS	34	25	9	5	297	1822	10.7	9.7	60	0.1

The subgrade is also evaluated for field densities and moistures as well as index properties as illustrated in Table E15 and Table E16.

Table E15 Test Results for Field Densities for Top Subgrade Layer

TP	Chainage	FDD Kg/m^3	PMC %	MDD - T99 Kg/m3	OMC %	Relative Compaction %	Relative Moisture Content %
1	2+215	1779	8.1	1720	8.1	103	89
2	23+750	1560	13.8	1624	12.6	96	110
3	42+120	1516	11	1684	9.8	90	110
4	51+150	1530	8.1	1670	7.4	92	109
5	62+590	1642	12.4	1785	11.5	92	108
6	96+600	1732	12.9	1755	14.4	99	90
7	102+510	1785	11.2	1748	13.4	102	84
8	113+210	1834	10.7	1717	11.1	101	96

Table E16 Summary of Index Properties for the Subgrade Layers

TP	Chainage	Offset	LL	PL	PI	LS	PM	MDD	OMC	NMC	CBR	Swell
1	2+215	TSG	NON - PLASTIC				0	1720	9.1	7.9	20	0.1
		BSG	NON - PLASTIC				0	1710	9.6	8.3	10	0.1
		NATIVE	NON - PLASTIC				0	1642	11.3	10.9	8	0.1
2	23+750	TSG	NON - PLASTIC				0	1624	12.6	13.4	40	0.1
		BSG	NON - PLASTIC				0	1680	9.9	11.1	9	0.1
		NATIVE	NON - PLASTIC				0	1654	10.2	10.4	4	0.1
3	42+120	TSG	30	14	16	8	1232	1684	9.8	10.8	11	0.1
		BSG	31	15	16	8	1104	1637	10	10.9	7	0.1
		NATIVE	33	18	18	9	1188	1665	10.9	12.1	6	0.1
4	51+150	TSG	33	18	16	8	1088	1670	7.4	8.3	13	0.1
		BSG	30	16	14	7	1022	1606	9.4	10.8	8	0.1
5	62+590	TSG	28	17	11	5	891	1785	11	12.1	12	0.1
		BSG	29	23	6	3	492	1744	9.8	10.8	10	0.1
6	96+600	TSG	35	20	15	8	825	1755	14.4	13.6	15	0.1
		BSG	31	13	18	9	1080	1722	12.4	11.9	11	0.1
		NATIVE	34	15	19	9	1102	1658	11.1	9.9	7	0.1
7	102+510	TSG	34	20	14	7	644	1748	13.4	12.3	14	0.1
		BSG	32	19	13	6	559	1754	13.3	13	13	0.1
		NATIVE	31	15	16	8	944	1720	11.4	25.9	6	0.1
8	113+210	TSG	33	21	12	6	576	1817	11.1	10.5	18	0.1
		BSG	36	21	15	8	795	1785	11.8	9.7	15	0.1
		NATIVE	NON-PLASTIC				0	1735	9.9	11.9	8	0.1

The next stage is to determine structural numbers from pavement layer thicknesses and layer strength coefficients using the methods given in Section 9.7. The same can also be obtained using the DCP but only for unbound or slightly bound layers, Table E17 and Table E18.

Table E17 Modified Structural Number from the Trenching Data

HS	Trenching Location	Modified Structural
1	2+215	3.9
2	23+750	4.2
3	42+120	3.2
4	51+150	3.7
5	62+590	3.9
6	96+600	3.8
7	102+510	3.9
8	113+210	4.0

Table E18 Summary of DCP Data Analysis

TP	Chainage (km)	Base Thickness mm	Base CBR %	Sub-base Thickness (mm)	Subbase CBR %	Subgrade CBR	Structural Number (SN)	Modified Structural Number (SNP)
1	2+215	137	213	233	488*	50	1.84	3.81
2	23+768	190	123	221	89	24	2.04	3.77
3	42+120	140	231	161	84	10	1.48	2.81
4	51+150	161	76*	224	21	24	1.53	3.26
5	62+590	250	106	195	266*	50	2.26	4.36
6	96+600	305	38*	215	53	40	1.95	3.91
7	102+510	196	158	176	56	36	1.84	3.87
8	113+200	213	38*	212	63	50	1.63	3.67
	Average	**199**	**166**	**204**	**61**	**36**	**1.8**	**3.7**

**Shows that the values were abnormal and thus they were not used for the analysis*

The next stage is to analyse the deflections. Figure E1 shows that the normalised deflections against chainages. The normalised deflections are then used to determine uniform sections using CUMSUMS method described in Chapter 9, see Figure E2.

Table E1 Deflection Test Results

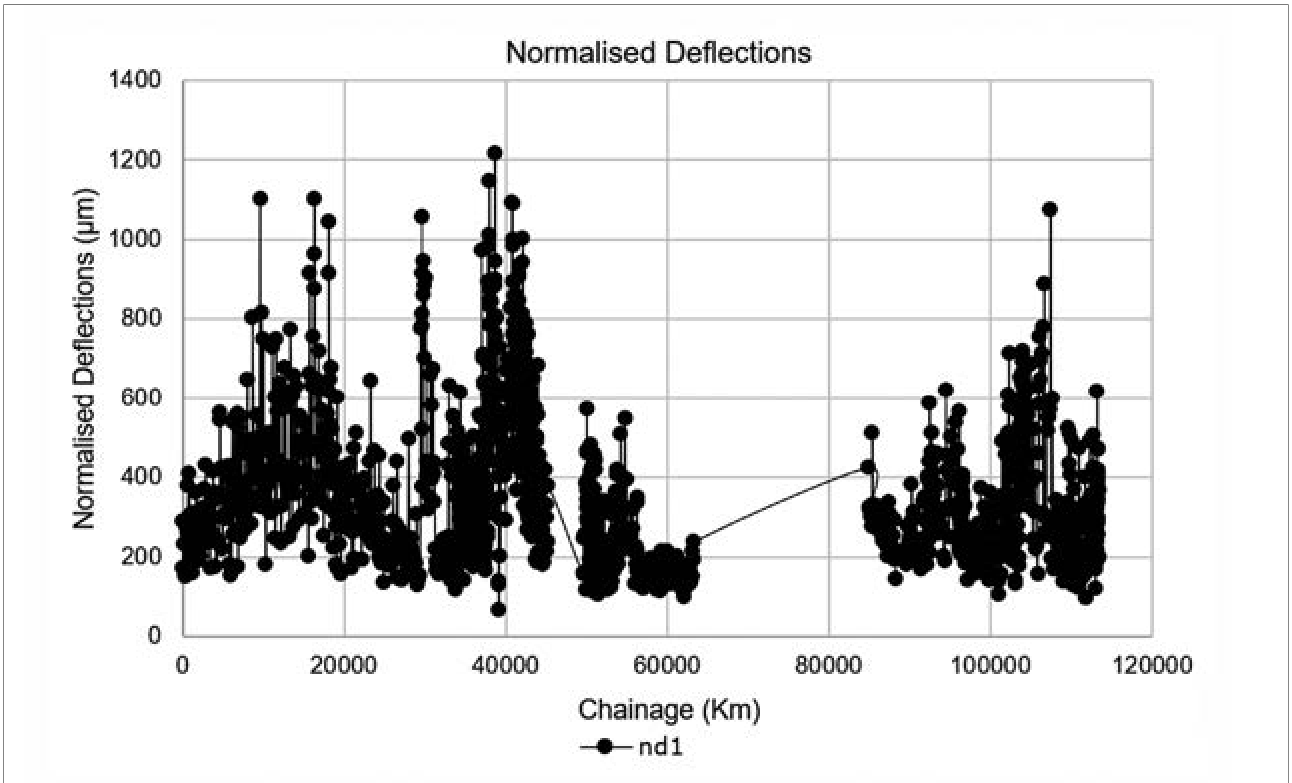
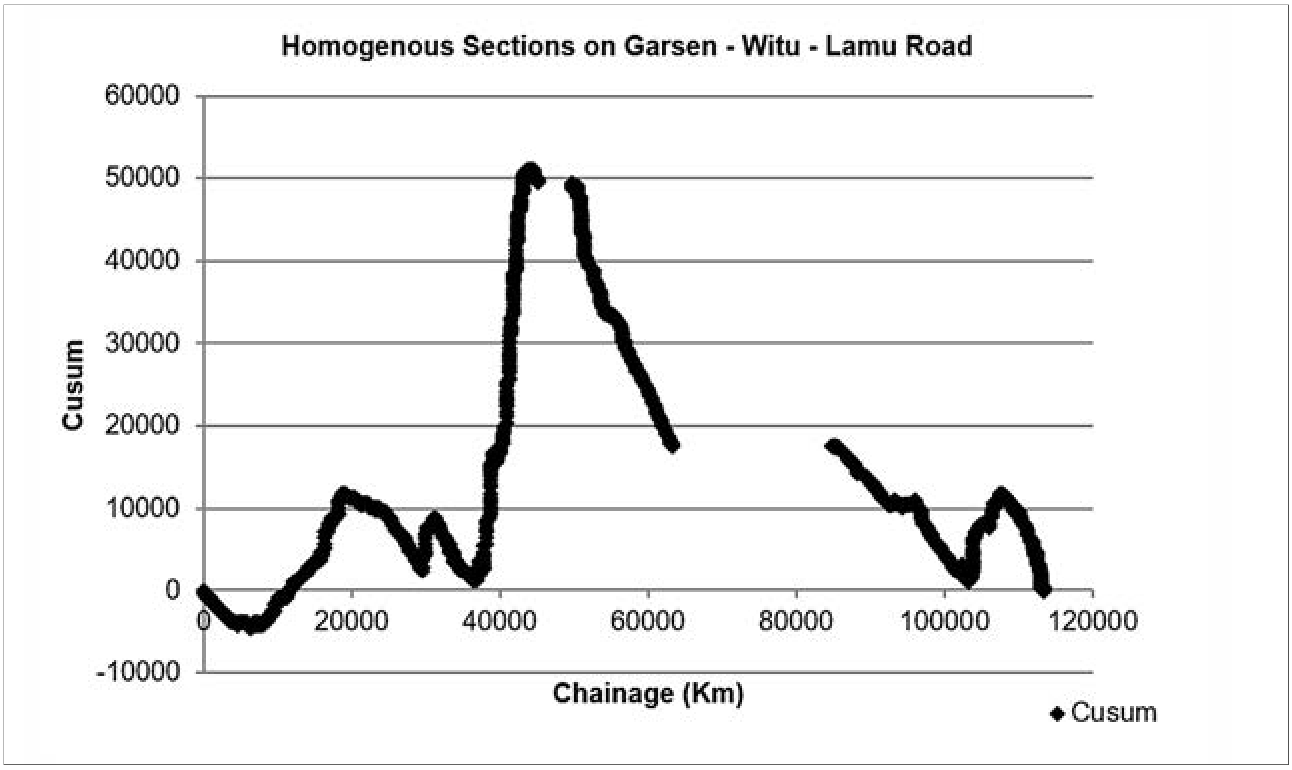


Table E2 Determination of Uniform Sections



From the CUSUM computation and graphs in Figures E1, and E2 the road was divided into the following homogenous sections:

i. Km 0+000 – 5+200 = 5.2 km
ii. Km 5+200 – 18+298 = 13.1 km
iii. Km 18+298 – 28+899 = 10.6 km
iv. Km 28+899 – 30+801 = 1.9 km
v. Km 30+801 – 36+016 = 5.2 km
vi. Km 36+016 – 45+200 = 9.2 km
vii. Km 49+650 – 63+304 = 13.7 km
viii. Km 84+915 – 102+801 = 17.9 km
ix. Km 102+801 – 107+200 = 4.4 km
x. Km 107+200 – 113+465 = 6.3 km

The next stage is to determine the mean deflections for each uniform section Table E19. These deflections should be normalised to a standard temperature as described in Chapter 9 and the results are illustrated in Table E20. The mean deflections are presented in the form of deflection bowls in Figure E3.

Table E19 Mean Deflections on Homogeneous Sections

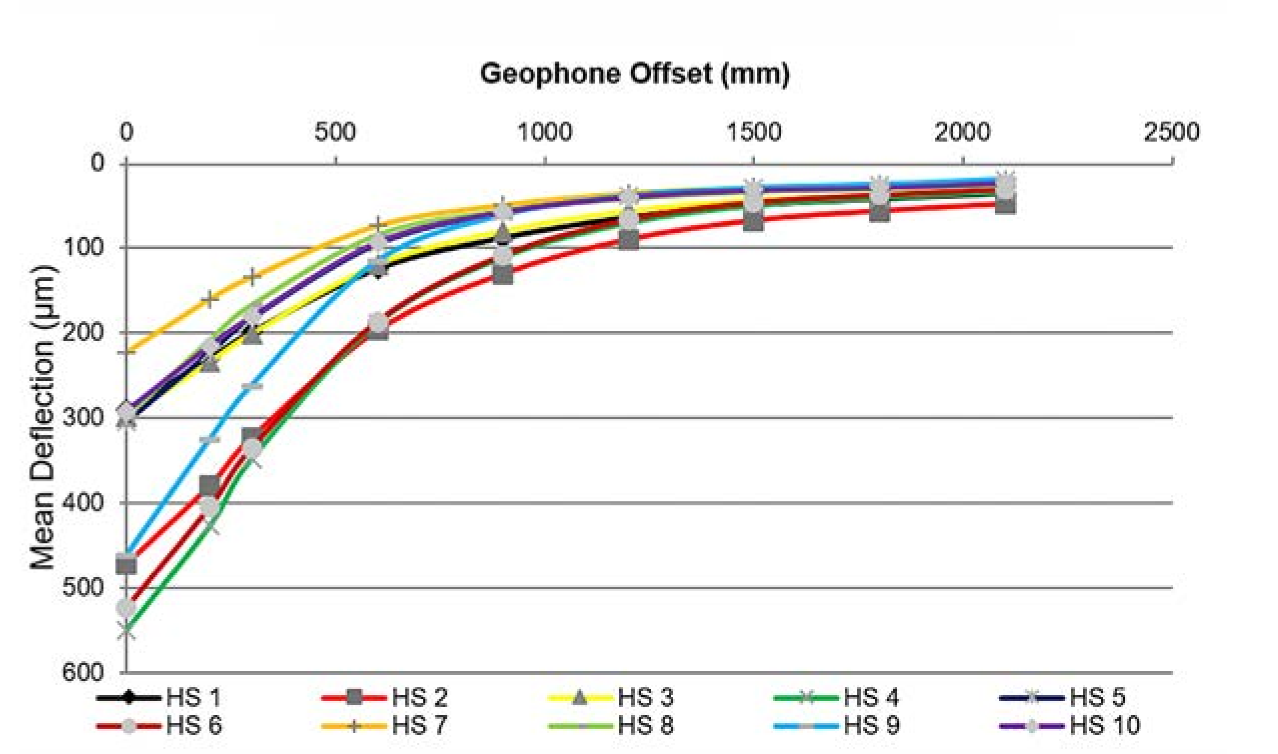
Homogenous Section (HS)	Length (Km)	Geophone Offsets mm								
		0	200	300	600	900	1200	1500	1800	2100
		Mean Normalised Deflections in µm								
0+000 – 5+200	5.2	290	230	198	125	88	64	50	42	36
5+200 – 18+298	13.1	472	379	323	197	130	90	67	56	48
18+298 – 28+899	10.6	298	233	200	119	80	56	43	37	30
28+899 – 30+801	1.9	550	427	348	188	111	70	50	41	34
30+801 – 36+016	5.2	305	223	182	95	58	38	29	25	19
36+016 – 45+200	9.2	524	404	335	186	109	67	46	37	31
49+650 – 63+304	13.7	223	160	134	73	49	35	27	24	19
84+915 – 102+801	17.9	299	208	167	84	56	41	33	30	24
102+801 – 107+200	4.4	461	325	261	115	61	39	28	24	18
107+200 – 113+465	6	292	215	180	93	57	40	31	28	22
Overall Mean	**87.5**	**365**	**274**	**228**	**125**	**78**	**52**	**39**	**34**	**27**

Table E20 Statistical Analysis of Normalised Deflection Data (nd)

HS	Homogenous Section (HS)	Min nd1	Max nd1	Mean nd1	Standard Deviation nd1
1	0+000 – 5+200	150	565	290	94
2	5+200 – 18+298	154	1103	472	186
3	18+298 – 28+899	136	679	298	110
4	28+899 – 30+801	130	1057	550	304
5	30+801 – 36+016	118	674	305	131
6	36+016 – 45+200	68	1216	524	214
7	49+650 – 63+304	100	574	223	96
8	84+915 – 102+801	105	713	299	102
9	102+801 – 107+200	132	888	461	161
10	107+200 – 113+465	98	1075	292	124
	Overall Mean	**68**	**1216**	**365**	**190**

1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 Appendices

Figure E3 Deflection Bowls of Homogeneous Sections



The E-moduli are then determined through back calculation and this can be done manually or through use of software. The results are illustrated in Table E21.

Table E21 Pavement Layers Moduli on Homogeneous Sections

SN	HS	Mean AC Moduli (MPa)	Mean Base Moduli (MPa)	Mean Subbase Moduli (MPa)	Mean Subgrade Moduli (MPa)
1	0+000 – 5+200	18728	2518	1603	180
2	5+200 – 18+298	13230	1456	1202	117
3	18+298 – 28+899	20535	2365	1326	206
4	28+899 – 30+801	13667	1464	1197	147
5	30+801 – 36+016	14048	1927	819	303
6	36+016 – 45+200	10572	1283	919	140
7	49+650 – 63+304	20834	2760	2635	341
8	84+915 – 102+801	17780	1817	637	313
9	102+801 – 107+200	10894	1317	495	211
10	107+200 – 113+465	10921	2210	601	278
	Average	**15138**	**1901**	**1174**	**236**

When using the structural number method, the modified structural number should be determined, Table E22. The overlay thickness can now be determined from the SN deficit i.e., the difference between the Required SN_d and SNP as described in Section 9.7. The strengthening/overlay required is illustrated in Table E23.

Table E22 Pavement Layer Indices

SN	HS	Mean AC Moduli (MPa)	Mean Base Moduli (MPa)	Mean Subbase Moduli (MPa)	Mean Subgrade Moduli (MPa)
1	0+000 – 5+200	91	134	28	5.9
2	5+200 – 18+298	149	233	42	4.7
3	18+298 – 28+899	97	144	26	5.9
4	28+899 – 30+801	202	278	37	4.9
5	30+801 – 36+016	122	145	18	6.1
6	36+016 – 45+200	189	269	36	4.6
7	49+650 – 63+304	89	99	16	7.0
8	84+915 – 102+801	132	126	17	6.0
9	102+801 – 107+200	200	223	20	4.9
10	107+200 – 113+465	111	141	17	6.1
	Average	**137**	**175**	**25**	**5.7**

When using the software with inputs of the layer thicknesses of the existing pavement which are fixed and the layer moduli which are determined through back calculation, with a fixed E-modulus value for the new layer considered for overly its thickness can be determined and results would be as illustrated in Table E23.

Table E23 Overlay Requirements for the Homogenous Sections

HS	Homogenous Section (HS)	Average of Reinforcement (mm)	Average of ResLifeNow (Years)
1	0+000 – 5+200	1	20
2	5+200 – 18+298	15	16
3	18+298 – 28+899	4	19
4	28+899 – 30+801	36	11
5	30+801 – 36+016	9	17
6	36+016 – 45+200	25	13
7	49+650 – 63+304	2	19
8	84+915 – 102+801	21	14
9	102+801 – 107+200	28	12
10	107+200 – 113+465	11	17
	Average	**15**	**16**