



Federal Republic of Nigeria

Federal Ministry of Works

Highway Manual Part 1: Design

Volume III:
Pavements and Materials Design

2013

FOREWORD

The vision statement of the Federal Ministry of Works is to elevate Nigerian roads to a standard where they become National economic and socio-political assets, contributing to the Nation's rapid growth and development, and to make Federal roads functional, safe, pleasurable, and an avenue for redeeming Nigerians' trust and confidence in Government. This vision statement is in tune with the Transformation Agenda of the President of the Federal Republic of Nigeria, His Excellency, Dr Goodluck Ebele Jonathan, GCFR. Based on the foregoing, our mission is to use the intellectual, management and material resources available to the Ministry to make Nigerian roads functional all the time. The principal goal of the Ministry is to drive the transformation agenda by improving road transport infrastructure for the overall socio-economic derivable benefits and development of our great country, Nigeria.

In exercising this mission and in discharging its responsibilities, the Ministry identified the need for updated and locally relevant standards for the planning, design, construction, maintenance and operation of our roads, in a sustainable manner. One of the main reference documents for this purpose is the Highway Manual, which previously included Part 1: Design and Part 2: Maintenance. Both current parts of the Highway Manual were first published in 1973 and 1980 respectively and have been subjected to partial updating at various times since then. The passage of time, development in technology, and a need to capture locally relevant experience and information, in the context of global best practices, means that a comprehensive update is now warranted.

The purpose of the Highway Manual is to establish the policy of the Government of the Federal Republic of Nigeria with regard to the development and operation of roads, at the Federal, State and Local Government levels, respectively. In line with this objective, the Manual aims to guide members of staff of the Ministry and engineering practitioners, with regard to standards and procedures that the Government deem acceptable; to direct practitioners to other reference documents of established practice where the scope of the Manual is exceeded; to provide a nationally recognized standard reference document; and to provide a ready source of good practice for the development and operation of roads in a cost effective and environmentally sustainable manner.

The major benefits to be gained in applying the content of the Highway Manual include harmonization of professional practice and ensuring uniform application of appropriate levels of safety, health, economy and sustainability, with due consideration to the objective conditions and needs of our country.

The Manual has been expanded to include an overarching Code of Procedure and a series of Volumes within each Part that cover the various aspects of development and operation of highways. By their very nature, the Manual will require periodic updating from time to time, arising from the dynamic nature of technological development and changes in the field of Highway Engineering.

The Ministry therefore welcomes comments and suggestions from concerned bodies, groups or individuals, on all aspects of the document during the course of its implementation and use. All feedback received will be carefully reviewed by professional experts with a view to possible incorporation of amendments in future editions.



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Honourable Minister

Federal Ministry of Works,

Abuja, Nigeria

May, 2013

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1 Introduction

1.1 Description of the Manual

1.1.1 Introduction to the Manual

The Highway Manual aims to guide members of staff of the Ministry and engineering practitioners, with regard to standards and procedures that the Government deems acceptable for the planning, design, construction, maintenance, operation and management of roads. The Manual directs practitioners to other reference documents of established practice where the scope of the Manual is exceeded; provides a nationally recognized standard reference document; and provides a ready source of good practice for the development and operation of roads in a cost effective and environmentally sustainable manner.

1.1.2 Arrangement of the Manual

The Highway Manual comprises a Code of Procedure and two parts, each of which has been divided up into separate volumes, in the manner shown in Figure 1.1.

1.2 Overview of Volume III: Pavements and Materials Design

1.2.1 General

Volume III of the Highway Manual Part 1: Design deals with the Pavement and Materials Design for highways.

1.2.2 Purpose

The purpose of this volume is to give guidance and recommendations to the engineers responsible for the design of pavements and materials for Federal Highways in Nigeria.

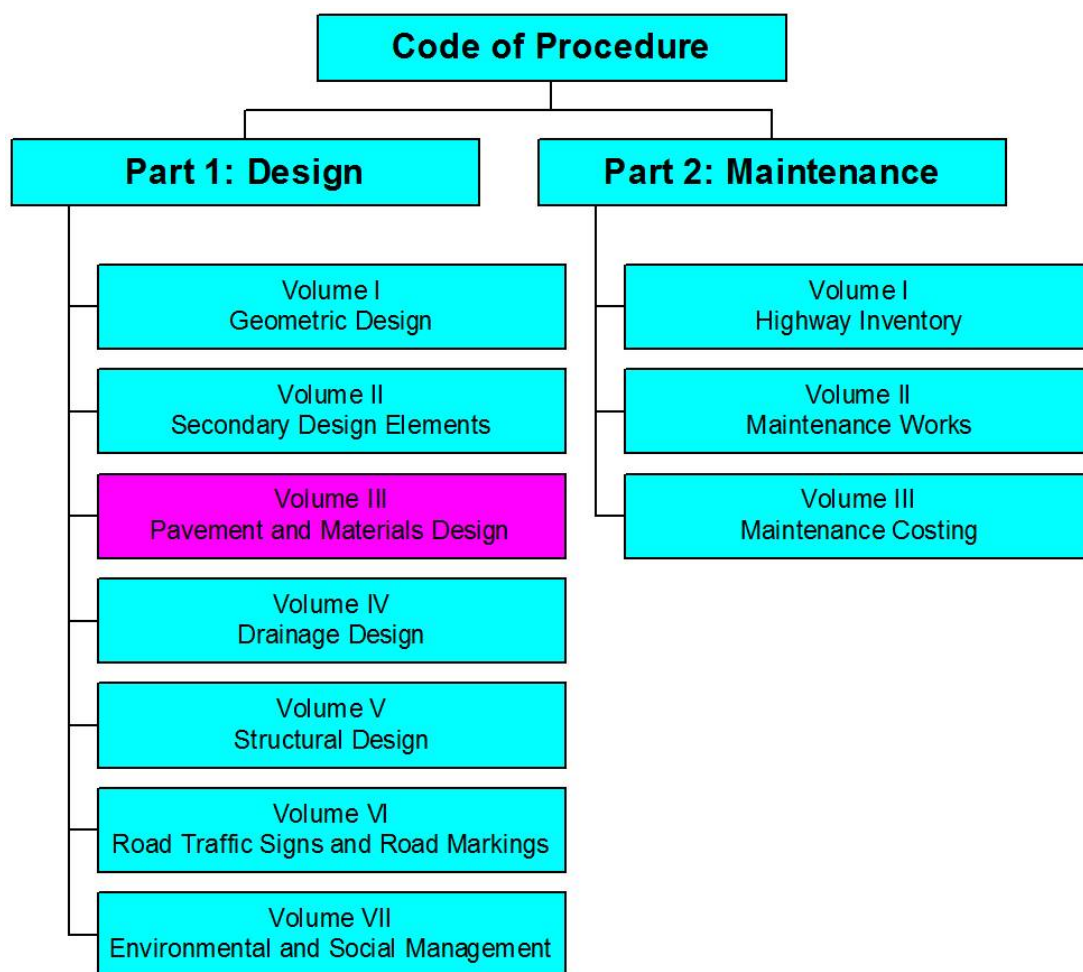


Figure 1.1 Arrangement of Volumes in the Highway Manual

1.2.3 Scope of this Volume

This volume has been developed by the Federal Ministry of Transportation (Works) with the intent to reflect policy and establish uniform policies and procedures for planning and designing highways. The procedures presented in this volume are applicable to all classes of roads in Nigeria.

The contents of the volume are partly guidelines and recommendations and partly standards which as a general rule should be adhered to. The information, guidance and references contained in this volume are not intended as a substitute for sound engineering judgment. It should be recognized that situations may be encountered during the design of highways that are beyond the scope of this volume. Numerous sources of comprehensive information are listed at the end of the volume; these sources should be used to supplement the information contained in this volume. In some instances, special conditions may require the use of other references and/or

standards and the use of these standards can only be sanctioned by the Federal Ministry of Works.

1.3 Pavement Terminology

Figure 1.2 Illustrates the main pavement terminology used in this guideline document.

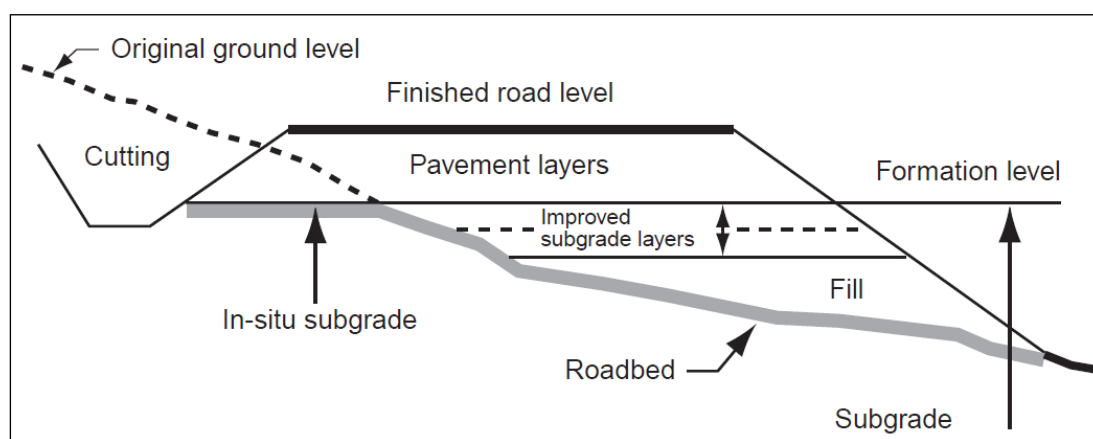


Figure 1.2 Pavement Terminology

1.4 Design Philosophy and Process

The structural design of pavements aims to protect the subgrade from traffic loads, by providing pavement layers which will achieve a chosen level of service, with maintenance and rehabilitation during the analysis period, as cost-effectively as possible.⁽¹⁾

1.4.1 Service Objective (SO)

When the need for accessibility or traffic capacity improvement in a certain area has been identified by the responsible authority, two basic decisions need to be taken by management in order to provide the necessary directive and inputs for the design process, namely:

- The functional service level of the road or facility improvement
- The analysis period over which the service is anticipated.

These inputs and directives are called the Service Objective (SO) of the project, taking into account such aspects as the importance of the road link, riding quality, safety, traffic capacity, funding, etc. The SO largely determines the standard of the geometrical and structural designs for that particular road link.

1.4.2 Functional Service Level (FSL)

A distinction is made between the functional requirements and the structural requirements of a road link. The functional requirements relate to the functional service which the road has to deliver in order to fulfil the need as defined by the Service Objective (SO). The structural requirements, however, relate to the support (i.e. bearing capacity) necessary to guarantee the functional service at a given design reliability.

The Functional Service Level (FSL) is the qualitative measure for operating conditions on a given portion of a road, and is related to the perceptions of motorists of those conditions. It is basically determined by factors such as speed, travelling time, delays, freedom to change position in the traffic stream, safety and driving comfort.

1.4.3 Analysis Period (AP)

The Analysis Period (AP) is usually equal to the functional period for which the road will have to deliver its functional service. The AP may be made up of one or more Structural Design Periods (SDP), each with its own Life Cycle Strategy (LCS).

1.4.4 Structural Design Period (SDP)

The Structural Design Period (SDP) is defined as the period during which it is predicted that no structural improvements will be required, linked to a specified design reliability. To select the “optimum pavement” in terms of present worth of cost, it is necessary to evaluate the Life Cycle Strategy of the different pavement structures.

1.4.5 Life Cycle Strategy (LCS)

The Life Cycle Strategy (LCS) for any pavement design incorporates the predicted maintenance and rehabilitation programme for that pavement based on its anticipated behaviour under the prevailing conditions. It normally includes the funding needs programme for the specific road.

The design process is the first step in the life cycle of a pavement. Not only will the final pavement design have an influence on the behaviour of the pavement, but it also lays the foundation for the maintainability (i.e. frequency and type) and salvage value of the pavement when it has to be rehabilitated or reconstructed. Thus the LCS considers the overall performance of the pavement, both structurally and economically, over its structural design and analysis period.

1.4.6 Structural Objective (STO)

When a road pavement is designed initially, the design should agree with the current service objective, and with due regard for the life cycle strategy for that section of road.

The aim of the basic structural objective (STO) may be summarised as follows:

To produce a structurally balanced pavement structure of sufficient bearing capacity under the prevailing environmental conditions, in order to fulfil the function need as defined by the Functional Service Level (FSL). This includes the design and maintenance predicted in the Life Cycle Strategy (LCS) that it will be able to carry the traffic cost effectively, over the Structural Design Period (SDP) in accordance with the Service Objective (SO).

While the pavement may be maintained or upgraded in accordance with the life cycle strategy in order to uphold the functional serviceability commensurate with the service objective over the analysis period, it should not exhibit signs of major distress requiring structural rehabilitation. This means that the Present Worth of Cost (PWOC) of alternative designs should be calculated during the life cycle strategy analysis, in order to determine the most economical pavement structure integrated with the in situ conditions.

1.4.7 Road Category

Generally, a road authority may have a number of road categories to suit the different levels of service the system has to deliver, based on the associated service objectives. Each of these road categories will necessitate certain geometrical and structural standards to ensure that the service objectives of the road can be met, and maintained throughout its analysis period. The more important a road, the

higher its level of service and thus its physical properties and standards, hence these roads have a reduced risk of failure (i.e. higher design reliability) over the structural design period. Four typical road categories may be considered in the context of pavement design. These are as shown on Table 1.1

Table 1.1: Definition of Typical Road Categories

	Road Category			
	A	B	C	D
Description	Major inter-urban freeways and major rural roads	Inter-urban collectors and rural roads	Lightly trafficked rural roads, strategic roads	Rural access roads
Importance	Very important	Important	Less important	Less important
Service Level	Very high level of service	High level of service	Moderate level of service	Moderate to low level of service

1.4.8 Abbreviations

A

AADT Average Annual Daily Traffic

AASHTO American Association of State Highway and Transportation Officials

ADT Average Daily Traffic

AP Analysis Period

B/C

CBR California Bearing Ratio

D

DCP Dynamic Cone Penetrometer

DT Design Traffic

E**EF** Equivalence Factors**ESA** Equivalent Standard Axles**F/G****FSL** Functional Service Level**J/K/L****LCS** Life Cycle Strategy**M/N****MDD** Maximum Dry Density**O/P****OMC** Optimum Moisture Content**PFA** Pulverised Fuel Ash**PI** Plasticity Index**PL** Plastic Limit**PWOC** Present Worth of Cost

S

SA	Standard Axles
SDP	Structural Design Period
SO	Service Objective
SI	International System
STO	Structural Objective

T

TRL	Transport Research Laboratory (UK)
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U

UCS	Unconfined Compressive Strength
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V/W

WIM	Weigh in motion
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1.4.9 Definitions

A

Aggregate

Hard mineral elements of construction material mixtures, for example: sand, gravel (crushed or uncrushed) or crushed rock.

Analysis period

A selected period over which the present worth of construction costs, maintenance costs (including user costs) and salvage value are calculated for alternative designs and during which full reconstruction of the pavement is undesirable.

Average Annual Daily Traffic (AADT)

Total yearly traffic volume in both directions divided by the number of days in the year.

Average Daily Traffic (ADT)

Total number of vehicles (traffic volume) during a given time period in whole days greater than one day and less than one year divided by the number of days in that time period.

B

Base Course

The layer(s) of a pavement placed directly upon the sub grade or subbase of planned thickness.

Behaviour

A function of the condition of the pavement over time.

Bituminous Surface Treatment

The mixing of a bituminous binder material with a specified depth of roadbed material then spreading and compacting the mixture.

Borrow

Material not obtained from roadway excavation but secured by widening cuts, flattening cut back slopes, excavating from sources adjacent to the road within the right-of-way, or from selected borrow pits as may be noted on the plans.

C***Camber***

The slope from a high point (typically at the center line of a road) across the lanes of a highway. It is also called Cross fall.

Capacity

The maximum number of vehicles that can pass a point on a road or a designated lane in one hour without the density being so great as to cause unreasonable delay or restrict the driver's freedom to maneuver under prevailing roadway and traffic conditions.

Capillary Moisture

Moisture which clings to the soil particles by surface tension and reaches the particles either when free water passes through the soil or by capillary action from a wetter stratum. Within limits, it can move in any direction.

Capping Layer

A layer of selected fill material placed on the topmost embankment layer or the bottom of excavation.

Carriageway

Part of the roadway including the various traffic lanes and auxiliary lanes but excluding shoulders.

Centre Lane

On a dual three-lane road, the middle lane of the three lanes in one direction.

Centreline

Axis along the middle of the road.

Construction Joint

A joint made necessary by a prolonged interruption in the placing of concrete.

Contraction Joint

A joint normally placed at recurrent intervals in a rigid slab to control transverse cracking.

Criterion

A yardstick according to which some or other quality of the road can be measured. Guideline values are specific numerical values of the criterion.

Crossfall

The tilt or transverse inclination of the cross-section of a carriageway which is not cambered, expressed as a percentage.

D**Density**

The number of vehicles per kilometer on the travelled way at a given instant. (Hence Average Volume = Average Density x Average Speed.)

Design CBR of subgrade

The representative laboratory California Bearing Ratio value for the subgrade which is used in the structural design.

Design Period

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

Deformed Bar

A reinforcing bar for rigid slabs conforming to "Requirements for Deformations" in AASHTO Designations M 31M.

Distress

The visible manifestation of the deterioration of the pavement with respect to either the serviceability or the structural capacity.

Dowel

A load transfer device in a rigid slab, usually consisting of a plain round steel bar.

Dual Carriageway Road

A road in which there are two physically separated carriageways reserved for traveling in opposite directions

E**Embankment**

That portion of the road prism composed of approved fill material, which lies above the original ground and is bounded by the side slopes, extending downwards and outwards from the outer shoulder breakpoints and on which the pavement is constructed.

Equivalent Standard Axles (ESAS)

Summation of equivalent 8.2 metric ton single axle loads used to combine mixed traffic to design traffic for the design period (cf. ESA's Pavement Design Manual, Volume 1).

Expansion Joint

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

F**Fill**

Material which is used for the construction of embankments. 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.

Flat (Terrain)

Flat terrain with largely unrestricted horizontal and vertical alignment; transverse terrain slope up to 5 percent.

Flexible Pavements

Pavements having sufficient low bending resistance to maintain intimate contact with the underlying structure yet having the required stability furnished by aggregate inter lock, particle friction and/or surface tension to support the traffic; e.g. macadam crushed stone, gravel, and all bituminous types not supported on a rigid base.

G***Geometric design***

The design of the geometry of the road surface for traffic flow and for the safety and convenience of the road user.

Ground Water

Free water contained in the zone below the water table,

H***Heavy vehicle***

A vehicle with an axle load > 4000 kg, usually with dual rear wheels.

I***In situ Layer***

The material in excavations, embankments and embankment foundations immediately below the first layer of subbase, base, or pavement, and to such depth as may affect the structural design.

J/K/L***Longitudinal Joint***

A joint normally placed between traffic lanes in rigid pavements to control longitudinal cracking.

M***Maintenance***

Routine work performed to keep a pavement, under normal conditions of traffic and forces of nature, as nearly as possible in its as-constructed condition.

Mechanistic analysis

Analysis of a system taking into account the interaction of various structural components as a mechanism, here used to describe a design procedure based on fundamental theories of structural and material behaviour in pavements.

Median

Area between the two carriageways of a dual carriageway road excluding the inside shoulders.

Modified material

A Material of which the physical properties have been improved by the addition of a stabilising agent but in which cementation has not occurred.

Mountainous (terrain)

Terrain that is rugged and very hilly with substantial restrictions in both horizontal and vertical alignment; transverse terrain slope 25-75 percent.

N/O/P

Pavement

A multi-layered horizontal structure which is constructed for the purpose of carrying traffic. Includes the layers of different materials which comprise the pavement structure.

Pavement Design

The arrangement of available materials in varying depths to achieve the most advantageous combination of foundation courses and pavement which will accommodate the anticipated wheel-load repetitions.

Pavement Layers

The layers of different materials, which comprise the pavement structure.

Performance

The measure of satisfaction given by the pavement to the road user over a period of time, quantified by a serviceability/age function.

Permeability

The property of soils which permits the passage of any fluid. Permeability depends on grain size, void ratio, shape and arrangement of pores.

Perched Ground Water

Ground water located above the level of the general body of ground water and separated from it by a zone of impermeable material.

Present Serviceability Index

Riding quality

Present worth of costs

Sum of the costs of the initial construction of the pavement, the later maintenance costs and the salvage value discounted to a present monetary value.

Project Specifications

The specifications relating to a specific project, which form part of the contract documents for such project, and which contain supplementary and/or amending specifications to the standard specifications.

Pumping

The ejection of foundation material, either wet or dry, through joints or cracks, or along edges of rigid slabs resulting from vertical movements of the slab under traffic.

R***Resurfacing***

A supplemental surface or replacement placed on an existing pavement to improve its surface integrity or increase its strength.

Riding quality

The general extent to which road users experience a ride that is smooth and comfortable or bumpy and thus unpleasant and perhaps dangerous.

Rigid Pavements

Pavements which due to high bending resistance distribute loads to the foundation over a comparatively large area: e.g., Portland cement and brick, stone block or bituminous pavement on a Portland cement concrete base.

Road

Way for vehicles and for other types of traffic which may or may not be lawfully usable by all traffic.

Road Bed

The natural in-situ material on which the embankment or capping layers are to be constructed.

Road Functional Classification

Classification of roads according to service provided in terms of the road hierarchy.

Road Prism

The cross sectional area bounded by the original ground level and the sides of slopes in cuttings and embankments excluding the pavement.

Road Width

A measurement at right angle to the centerline incorporating travelled way, shoulders and, when applicable, central reserve.

Roadside

General term denoting the areas adjoining the outer edges of the shoulders.

Roadway

Part of the road comprising the carriageway, shoulders and median. Also referred as the area normally travelled by vehicles and consisting of one or a number of contiguous traffic lanes, including auxiliary lanes and shoulders.

Roadway Width

The cross sectional area bounded by the original ground level and the sides of slopes in cuttings and embankments excluding the pavement.

Rolling (Terrain)

Terrain with low hills introducing moderate levels of rise and fall with some restrictions on vertical alignment; traverse terrain slope 5-20 percent.

Rural road

A surfaced secondary road serving small rural communities and carrying very light traffic with a relatively low level of service.

Reinforcement

Steel embedded in a rigid slab to resist tensile stresses and detrimental opening of cracks.

Rigid Pavement

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

Road Bed

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

Road Bed Material

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

S**Selected layer:**

The lowest of the pavement layers, comprising controlled material, either in situ or imported.

Serviceability

The measure of satisfaction given by the pavement to the road user at a certain time, quantified by factors such as riding quality and rut depth.

Shoulder

Part of the road outside the carriageway, but at substantially the same level, for accommodation of stopped vehicles for emergency use, and for lateral support of the carriageway.

Shoulder Breakpoint

The point on a cross section at which the extended flat planes of the surface of the shoulder and the outside slope of the fill and pavement intersect.

Stabilisation

The treatment of the materials used in the construction of the road bed material, fill or pavement layers by the addition of a cementations binder such as lime or Portland cement

or the mechanical modification of the material through the addition of a soil binder or a bituminous binder. Concrete and asphalt shall not be considered as materials that have been stabilised.

Standard Axle (SA):

80 kN Single axle dual wheel configuration is the Standard Axle (SA). (The maximum legally permissible single axle load (4 or more tyres) is 88 kN.)

Structural design

The design of the pavement layers for adequate structural strength under the design condition of traffic loading, environment and subgrade support.

Structural design period

The chosen minimum period during which the pavement is designed to carry the traffic in the prevailing environment with a reasonable degree of confidence that structural maintenance will not be required.

Structural maintenance

Measures that will strengthen, correct a structural flaw in, or improve the riding quality of an existing pavement, e.g. overlay, smoothing course and surface treatment, partial reconstruction (say base and surfacing), etc.

Subbase

One or more courses of soil or aggregate, or both, of planned thickness and quality placed on the sub-grade as the foundation for a base. Also in the case of rigid pavements, the layer below the concrete slab.

Subgrade

The completed earthworks within the road prism prior to the construction of the pavement layers. This comprises the in situ material of the roadbed and any fill material. In structural design only the subgrade within the material depth is considered.

Surfacing

The uppermost pavement layer which provides the riding surface for vehicles.

T***Tie Bar***

A deformed steel bar or connector embedded across a joint in a rigid slab to prevent separation of abutting slabs.

Traffic

Vehicles, pedestrians and animals travelling along a route.

Traffic Lane

Part of a travelled way intended for a single a stream of traffic in one direction, which has normally been demarcated as such by road markings.

Traffic Volume

The number of vehicles or persons that pass over a given section of a lane or a roadway during a time period of one hour or more.

Truck

A general term denoting a motor vehicle designed for transportation of property. The term includes single-unit trucks and truck combinations.

Truck Combinations

A truck tractor and semi-trailer, either with or without a full trailer, or a truck with one or more full trailers.

Two Axle Truck

A two axle freight vehicle with a total of six tyres. (Dual tyres on the rear axle.)

Typical Cross-Section

Cross-section of a road showing standard dimensional details and features of construction.

U/V***Volume***

The number of vehicles passing a given point during a specified period of time.

W***Water Table***

The surface of the ground water below which the void spaces are completely saturated.

Wearing Course

The top layer of a pavement designed to provide a surface resistant to traffic abrasion without necessarily imparting any structural values to the pavement. Wearing course includes light bituminous macadam (sometimes designated as armour coat), seal coat with mineral aggregate cover and coarse non-skid treatment similar to a seal coat.

Welded Wire Fabric

Welded steel wire fabric for concrete reinforcement.

2 Climate

The climatic conditions (moisture and temperature) under which the road will function must be taken into account in the design of a pavement structure.

The moisture conditions will largely determine the weathering of natural rocks, the durability of weathered natural road building and also, depending on drainage conditions, the stability of untreated materials in the pavement.

The climate may also influence the equilibrium moisture content, and the ambient pavement temperatures may affect the stability of bituminous surfacing.

The designer should therefore consider the climatic conditions and avoid using excessively water susceptible or temperature sensitive materials in adverse conditions.

2.1 Climatological Zones

Nigeria is divided into four distinct climatological zones, each with unique rainfall and temperature characteristics. Figure 2.1 shows the four zones on a state map of Nigeria.



Figure 2.1 Climatological Zones of Nigeria

Table 2.1 Describes the climate and rainfall range in each of the climatological zones.

Table 2.1 Climate and Rainfall Characteristics of Climatological Zones

Nigeria Climatological Zones		
Zone	Climate	Rainfall (mm)
Zone 1	Hot Dry	528 – 960
Zone 2	Temperate Dry	1 077 – 1 399
Zone 3	Hot Humid	1 183 – 1 787
Zone 4	Warm Humid	1 185 – 2 788

2.2 Temperature Distribution in Nigeria

Temperature is an important consideration in the selection of pavement materials, especially bituminous materials which can be highly temperature sensitive.

Figure 2.2 and Figure 2.3 show maps of the annual minimum and maximum temperature distribution across the country.

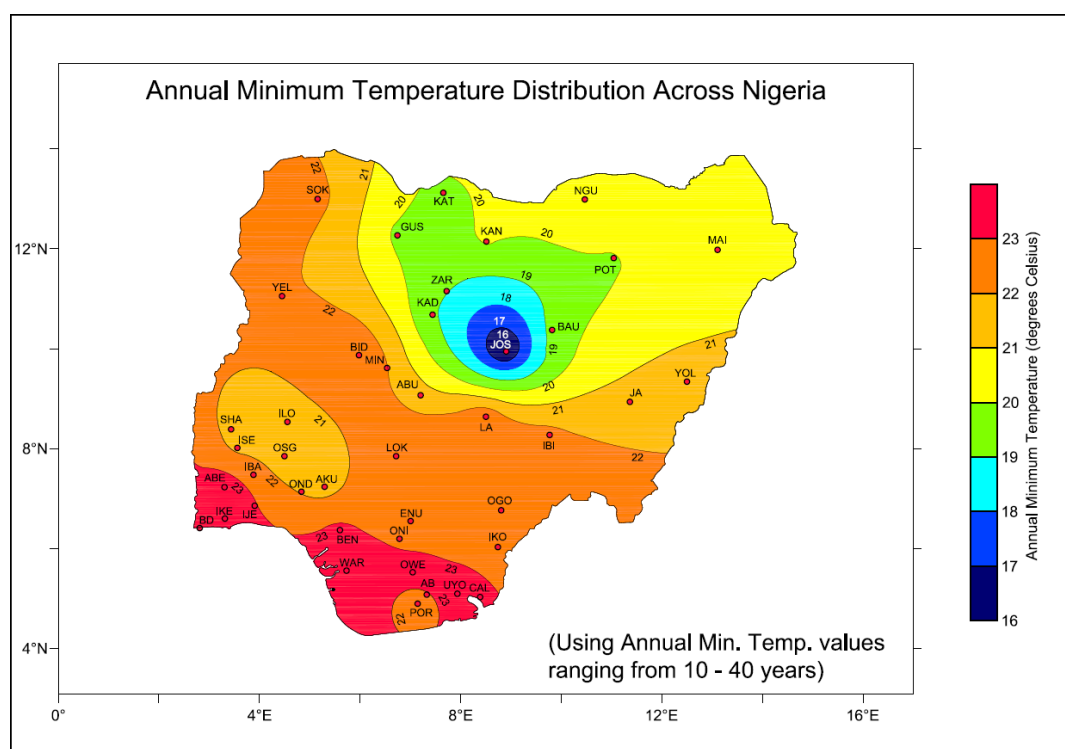


Figure 2.2 Annual Minimum Temperature Distribution

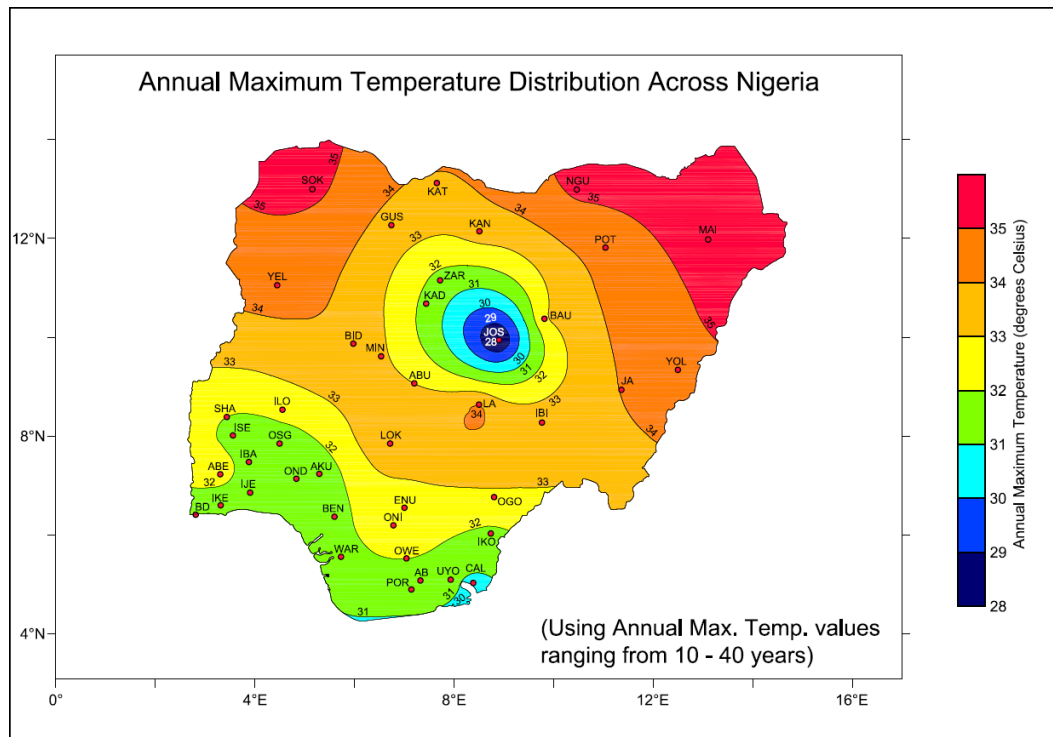


Figure 2.3 Annual Maximum Temperature Distribution

From the values shown on the map, it can be concluded that Nigeria has a generally moderate to hot temperature range. Provision should be made for high surface temperatures on the road surface, particularly in the northern regions.

2.3 Rainfall Distribution in Nigeria

Figure 2.4 shows the mean annual rainfall distribution across the country.

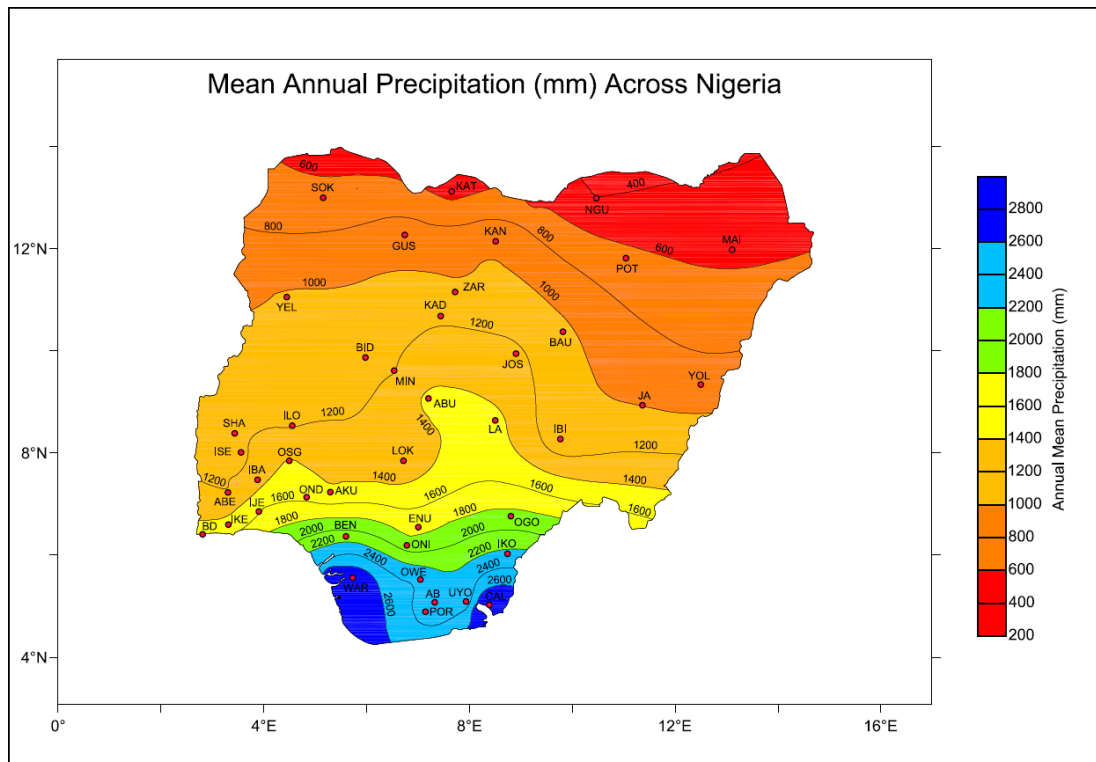


Figure 2.4 Annual Rainfall Distribution in Nigeria

From the values shown on the map, it can be concluded that the southern parts of Nigeria are fairly wet and pavements would be expected to perform in wet conditions often during the year. Special provisions should therefore be made for drainage design, including subsurface drainage.

The northern parts of the country are dry.

2.4 Definition of Wet or Dry Conditions

Rainfall can seasonably influence the bearing capacity of sub grade materials. Moisture has a direct effect on pavement wearing surfaces which will be reflected in the cost of maintenance and repairs.

Factors which will have an influence on the selection, apart from broad climatic considerations, also include drainage and maintenance regimes that are anticipated for the road. It is a basic fact that, for any road, the frequent ingress of water to the

pavement layers will result in unwanted deterioration under trafficking. The rate and degree of such deterioration will also therefore depend on the level of trafficking.

While the underlying requirement for any road is the provision of good drainage and operation of an effective maintenance programme to ensure that water does not penetrate the pavement, real life conditions may not always match these needs.

Although it is implicitly assumed that suitable drainage and maintenance should be effected during the life of the road, and that lack of either of these will undoubtedly have a negative impact on long-term performance, it is acknowledged that deficiencies do occur. Such deficiencies should, nevertheless, be addressed in order to preserve the investment made in the road.

3 Design Traffic and Pavement Class

3.1 General

The deterioration of paved roads caused by traffic, results from the magnitude of the individual wheel loads, the contact tyre pressure and the number of times these loads are applied (load repetitions). For pavement design purposes, it is necessary to consider not only the total number of vehicles that will use the road, but also the axle loads of these vehicles.

The loads imposed by light vehicles do not contribute significantly to the structural damage. For the purpose of structural design, cars and similar-sized vehicles can be ignored and only the axle loading of the heavy vehicles that will use the road during its design life need to be considered.

3.2 Structural Design Period (SDP)

The SDP is the period during which the road is expected to carry traffic at a satisfactory level of service, without requiring major rehabilitation or repair work.

For most road projects an economic analysis of between 10 and 20 years from the date of opening is appropriate, but for major projects, this period should be tested as part of the appraisal process.⁽²⁾

NOTE :

It is implicit, however, that certain maintenance work will be carried out throughout this period in order to meet the expected design life. This maintenance work is primarily to keep the pavement in a satisfactory serviceable condition, and would include routine maintenance tasks and periodic resealing as necessary. Absence of this type of maintenance would almost certainly lead to premature failure (earlier than the design life) and significant loss of the initial investment.

As described in section 1.4.5, a Life Cycle Analysis of a road project takes into consideration the initial capital required to construct the road, as well as the funding requirements for maintenance over the life of the road to maintain an adequate level of service. A Life Cycle Strategy is therefore normally the choice of high initial capital expenditure, with subsequent lower maintenance costs, or low initial capital expenditure, with subsequent higher maintenance cost.

The selection of Design Period is a matter of balance between the Life Cycle Strategy to be adopted, and the life cycle funding requirements. Design Period is therefore dependent on the quality, quantity and availability of resources, as well as political implications.

The selection of Design Period may be done on the basis of the category of the road, that is, its relative importance in the total road network, as measured by the volume and type of traffic that the road carries. The four road categories described in Table 1.3 may be considered as follows:

Category A

For Category A roads, the SDP should be relatively long because:

- These are normally the heaviest-trafficked roads in the country
- Road user costs are high and the cost of interrupting traffic for maintenance is high
- Road alignment is normally fixed with high certainty of not changing
- It is normally not acceptable to road users to carry out heavy rehabilitation at short intervals

Category B

For Category B roads, the SDP may vary depending on the circumstances. Long design periods will be selected when circumstances are similar to Category A roads. A shorter design period would be selected where:

- A short geometric life for a facility in a changing traffic situation
- There is a lack of funds
- There is a lack of confidence in design assumptions, especially the design traffic

Category C

A relatively short SDP is often selected for Category C roads because of financial constraints.

Category D

On Category D roads, expected traffic growth will be rapid or unpredictable. A relatively short structural period will enable maintenance and/or upgrading strategies to be adapted to circumstances without incurring a high initial capital outlay. The designs are initially more economical, but carry a relatively high risk of failure.

Table 3.1 shows the recommended design period for each road category.

Table 3.1 Typical Structural Design Periods for Various Road Categories

Road Category	Design Period
A	20
B	20
C	15
D	10

Note: An SDP longer than 20 years is not recommended, due to the difficulty of projecting traffic over a period that long.

3.3 Design Traffic

In Nigeria, the standard axle load is 80kN. (Legally permissible axle load is 8.2 tonnes)

The cumulative damaging effect of all individual axle loads is expressed as the number of equivalent 80kN single axle loads (ESAs or E80s). The ESAs thus represent the number of standard loads that would cause the same damage to the pavement, as the actual traffic spectrum of all the axle loads.

A pavement must be designed to have a specific bearing capacity which is expressed in terms of the number of Standard Axle (80kN) load repetitions that will result in a certain condition of deterioration - the terminal condition - indicating that the pavement has structurally “failed”, and can no longer support the functional service set by the service objective.

The pavement design process requires the estimation of the average daily number of ESAs on one lane at the opening of the new road to traffic, which is then projected and cumulated over the design period to give the design traffic loading.

This guideline document presents a procedure to calculate design traffic from available traffic information. For a detailed procedure for the collection of traffic information, including guidance on dealing with seasonal traffic variations, the designer is directed to the following publications:

- Overseas Road Note 40: A Guide to axle load surveys and traffic counts for determining traffic loading on pavements. ⁽³⁾
([http://www.transport-links.org/transport links/filearea/publications](http://www.transport-links.org/transport_links/filearea/publications))
- Highway Capacity Manual, 2010⁽⁴⁾
- NCHRP Report 538: traffic Data Collection, Analysis, and Forecasting for Mechanistic Pavement Design⁽⁵⁾

The estimation of Design Traffic involves various steps as described in sections 3.3.1 to 3.3.5

3.3.1 Baseline Traffic Flows

In order to determine the total traffic over the design life of the road, the first step is to estimate baseline traffic flows. The estimate should be the Annual Average Daily Traffic (AADT) or Average Daily Traffic (ADT) currently using the route (if an existing route), or expected to use the route on opening to traffic (if a new route), classified into the vehicle categories of cars, light goods vehicles, trucks (which normally includes several sub-classifications to differentiate rigid and articulated vehicles, trucks with trailers, and various multi-axle configurations typical to the area) and buses.

NOTE:

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

In order to reduce error, it is recommended that traffic counts to establish ADT at a specific site conform to the following practice:

- i. The counts are for seven consecutive days
- ii. The counts on some of the days are for a full 24 hours, with preferably one 24hour count on a weekday and one during a weekend. On the other days, 12 hour counts should be sufficient, with the 24hour counts used to determine an appropriate ratio to estimate the 24hour counts from the 12-hour counts.
- iii. Counts are avoided at times when travel activity is abnormal for short periods, for example, month-end, public holidays, etc.
- iv. If possible, the seven day counts should be repeated several times throughout the year

3.3.2 Determining Average Daily ESAs (ADE)

The traffic loading is calculated by converting the ADT to ADE. This is done by converting the volume of each vehicle class into Equivalent Standard Axles (ESAs).

The ADE is thus calculated as the sum of the product of the ADT per vehicle class, and the average ESA per vehicle class.

$$\text{ADE} = \sum \text{ADT}_j \times \text{E80}_j \text{Equation 3-1}$$

Where,

ADE = Average Daily ESAs

ADT_j = Average Daily Traffic per vehicle class 'j'

E80_j = Average ESA per vehicle class 'j'

The average ESAs per vehicle class are determined from axle mass surveys.

Based on an axle load study carried out in Nigeria in 2008, ESAs per heavy vehicle used for design should be as shown on **Table 3.2**

Table 3.2 Typical ESAs per Heavy Vehicle

Load-Control Situation	Range of ESAs per HV
No overloading	1,0 – 2,5
Overloading Expected	5,5 – 23,0

If no recent axle load data is available, it is recommended that axle load surveys of heavy vehicles are undertaken whenever a major road project is being designed. Ideally, several surveys at different periods which will reflect seasonal changes in the magnitude of axle loads are recommended.

Portable vehicle-wheel weighing devices are available which enable a small team to weigh up to 90 vehicles per hour. Detailed guidance on carrying out axle load surveys and analysing the results is given in Road Note 40.⁽³⁾

It is recommended that axle load surveys are carried out by weighing a sample of vehicles at the roadside. The sample should be chosen so that a maximum of about 60 vehicles per hour are weighed. The weighing site should be level and, if possible, constructed in such a way that vehicles are pulled clear of the road when being weighed. The portable weighbridge should be mounted in a small pit with its surface **level** with the surrounding area. This ensures that all of the wheels of the vehicle being weighed are level, and eliminates the errors which can be introduced by even a small twist or tilt of the vehicle. More importantly, it also eliminates the large errors that can occur if all the wheels on one side of multiple axle groups are not kept in the same horizontal plane. The load distribution between axles in multiple axle groups is often uneven, and therefore each axle must be weighed separately. The duration of the survey should be based on the same considerations as for traffic counting.

On certain roads, it may be necessary to consider whether the axle load distribution of the traffic travelling in one direction, is the same as that of the traffic travelling in the opposite direction. Significant differences between the two streams can occur on roads serving docks, quarries, cement works, etc, where the vehicles travelling one way are heavily loaded, but are empty on the return journey. In such cases, the results from the more heavily trafficked lane should be used when converting commercial vehicle flows to the equivalent number of standard axles for pavement design.

Similarly, special allowance must be made for unusual axle loads on roads which mainly serve one specific economic activity, since this can result in a particular vehicle type being predominant in the traffic spectrum. This is often the case, for example, in timber extraction areas, mining areas and oil fields.⁽⁶⁾

Two types of axle mass surveys may be done:

- Static weighing - this requires vehicles to be stationary
- Dynamic weighing- this is used at sites such as multi-lane highways or where the terrain and traffic flow does not allow for the static weighing of all the vehicles, or for a representative sample to be obtained.

The loads determined for each axle are converted to an equivalency factor using the equation:

$$F = \left(\frac{P}{80}\right)^n \text{Equation 3-2}$$

Where,

n = relative damage exponent

F = load equivalency factor

P = axle load, in kN

Note: The relative damage exponent, n, is an empirically determined number that is dependent on the type of pavement, its failure mechanism and its state.

3.3.3 Traffic Categories

In order to forecast traffic growth, it is necessary to separate traffic into three categories:

- Normal Traffic - Traffic which would pass along the existing road, even if no new pavement were provided.
- Diverted Traffic - Traffic that changes from another route (or mode of transport) to the project road because of the improved pavement, but still travels between the same origin and destination.
- Generated Traffic - Additional traffic which occurs in response to the provision of improvement of the road.

The growth in normal traffic is estimated on the basis of historic traffic growth rates. In the absence of historic information, a realistic growth rate is estimated on the basis of information available from other roads in other parts of the country or the expected economic development in the country.

Diverted Traffic and Generated Traffic are estimated through simplified traffic modelling, which might include an origin-destination survey on nearby parallel routes and assessment land-use in the vicinity of the new road.

3.3.4 Determination of Future Traffic Loading – ESA Growth Rate

The determination of traffic loading after the date on which the information was collected is done by projecting the initial ADE, using an appropriate growth rate. The ESA growth rate comprises two components:

- The increase in heavy vehicle traffic volume. This may be considered to consist of the overall traffic growth rate and the increase in heavy vehicles as a percentage of total traffic
- The increase in the loading of heavy vehicles

The ESA growth rate may be calculated from historical growth rates and by subjective adjustment by the designer. The designer should always critically evaluate growth rate figures that are obtained from whatever source, and consider whether the figures are realistic in the light of knowledge about local conditions. The following should be considered:

- Will facilities in the area generate additional heavy vehicle journeys? And if so, for how long?
- What economic growth is expected for the area?
- Are alternative modes of transport available, or will they be constructed?
- How could future government legislation affect heavy vehicle growth, e.g. deregulation and axle load limits?
- How much traffic will be diverted to the planned new route initially?
- Could the growth rate be negative?

A sensitivity analysis with different growth rates in E80s should be carried out.

3.3.5 Calculating Cumulative Equivalent Standard Axle Loading

The pavement design process requires the estimation of the average daily number of ESAs on one lane at the opening of the new road to traffic, which is then projected and cumulated over the design period to give the design traffic loading.

This is done as outlined in the following steps:

- i. Determine the baseline Average Daily Traffic (ADT) for each class of vehicle
- ii. Determine the **one-directional** traffic flow for each vehicle class expected over the design life and convert to ESAs using appropriate equivalency factors. (If detailed information is available that shows a difference between the flows in each direction, the higher of the two directional values should be used for design).
- iii. Project the ADE at a selected growth rate, cumulating the total over the design period to determine the design traffic load.

The cumulative ESA per lane may be calculated from:

$$ESA_{total} = ADE_{initial} \times f_y \dots \dots \dots \text{Equation 3-3}$$

Where,

f_y = cumulative factor from **Table 3.3**

y = structural design period

The design carriageway widths and type of road may be used to further analyse the probable design needs. Table 3.4 gives the basis for design traffic loading using the nominal totals for each direction as determined.

Table 3.3: Traffic Growth Factor

DESIGN PERIOD, Y (years)	Traffic Growth Factor, f_y I (% per annum)									
	2	4	6	8	10	12	14	16	18	20
4	1 534	1 612	1 693	1 776	1 863	1 954	2 048	2 145	2 246	2 351
5	1 937	2 056	2 181	2 313	2 451	2 597	2 750	2 912	3 081	3 259
6	2 349	2 518	2 699	2 892	3 098	3 317	3 552	3 801	4 067	4 349
7	2 768	2 998	3 248	3 517	3 809	4 124	4 465	4 833	5 229	5 657
8	3 195	3 498	3 829	4 193	4 592	5 028	5 506	6 029	6 601	7 227
9	3 632	4 017	4 446	4 923	5 452	6 040	6 693	7 417	8 220	9 110
10	4 077	4 558	5 100	5 711	6 399	7 174	8 046	9 028	10 131	11 370
11	4 530	5 119	5 793	6 562	7 440	8 444	9 589	10 895	12 385	14 082
12	4 993	5 704	6 527	7 481	8 586	9 866	11 347	13 062	15 045	17 336
13	5 465	6 312	7 305	8 473	9 846	11 458	13 352	15 575	18 184	21 242
14	5 947	6 944	8 131	9 546	11 232	13 242	15 637	18 491	21 887	25 928
15	6 438	7 601	9 005	10 703	12 757	15 240	18 243	21 873	26 258	31 551
16	6 939	8 285	9 933	11 954	14 434	17 478	21 213	25 796	31 415	38 300
17	7 450	8 996	10 916	13 304	16 279	19 984	24 599	30 346	37 500	46 398
18	7 972	9 735	11 957	14 763	18 308	22 790	28 459	35 625	44 681	56 115
19	8 504	10 504	13 062	16 338	20 540	25 934	32 859	41 749	53 154	67 776
20	9 046	11 304	14 232	18 039	22 996	29 455	37 875	48 852	63 153	81 769
21	9 599	12 136	15 473	19 877	25 697	33 398	43 594	57 091	74 951	98 561
22	10 163	13 001	16 788	21 861	28 668	37 815	50 113	66 650	88 873	118 711
23	10 739	13 900	18 183	24 004	31 937	42 762	57 545	77 737	105 300	142 892
24	11 326	14 836	19 661	26 319	35 532	48 302	66 018	90 598	124 685	171 908
25	11 925	15 809	21 227	28 818	39 486	54 507	75 676	105 517	147 559	206 728

$$f_y = 365 \times (1 + 0.01i)^y \times [(1 + 0.01i)^y - 1] / (0.01i)$$

Table 3.4: Factors for Design Traffic Loading

Road type	Design Traffic Loading	Comment
Single carriageway		
Paved road width 4,5 m or less	Up to twice the sum of the ESAs in each direction*	The total traffic must be designed for as there will be significant overlap in each direction. For widths of 3.5m or less, double the total should be used due to channelization
Paved road width 4.5 m to 6.0 m	80% of the sum of the ESAs in each direction	To allow for considerable overlap in the central section of the road
Paved road width more than 6.0 m	Total ESAs in the most heavily trafficked direction	No overlap effectively, vehicles remaining in lanes
Dual carriageway		
Less than 2,000 commercial vehicles per day in one direction	90% of the total ESAs in the direction	The majority of heavy vehicles will travel in one lane effectively
More than 2,000 commercial vehicles per day in one direction	80% of the total ESAs in the direction	The majority of heavy vehicles will still travel in one lane effectively, but greater congestion leads to more lane switching
* Judicious to use double the total ESAs expected, as normally these are low trafficked roads and this may give little difference in pavement structure.		

For dual carriageways it is not recommended to adopt different designs for the different lanes for the main reason that, apart from practical issues, there are likely to be occasions when traffic is required to switch to the fast lane or other carriageway due to repair works on the slow lane, for example. This could then lead to accelerated deterioration of the fast lanes, and any initial cost savings could be heavily outweighed by future expenditure and loss of serviceability.

3.4 Design Traffic Class

The pavement structures suggested in this guide are classified in various traffic categories by cumulative ESAs expected. Table 3.5 gives these classifications, and the design traffic determined from Section 3.3.5. This is used to decide which design pavement category is applicable.

Table 3.5 Design Traffic Classes

	Design Traffic Class Designation								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
Traffic ranges (million ESAs)	< 0,3	0,3 – 0,7	0,7 – 1,5	1,5 - 3	3 - 6	6 - 10	10 - 17	17 - 30	30 – 100

If calculated, design values are very close to the boundaries of a traffic class, the values used in the forecasts should be reviewed and a sensitivity analysis carried out to determine which category is most appropriate.

An important addition here is the inclusion of a new traffic category T9- traffic range 30 – 100 million ESAs. This category is not included in the design catalogue. If the design traffic is determined to be 'T9', the other methods contained in this catalogue should be used for design.

The lowest traffic class T1, for design traffic of less than 0,3 million ESAs, is regarded as a practical minimum, since realistic layer thicknesses, as well as materials specifications tend to rule out lighter structures for lesser traffic.

However, in the unlikely case that design traffic is estimated at less than 0,1 million ESAs (that is, traffic significantly less than the lowest class T1), the engineer should consider alternative designs proven locally for this very light traffic load.

Appendix A contains a summary of the axle load study carried out in Nigeria in 2008.

4 Subgrade

4.1 General

The type of subgrade is largely determined by the location of the road, that is, by the geology of the area traversed by the road.

The designer may refer to the following publications for information on Nigeria's geology:

- Akintola, F.A. (1982) Geology and geomorphology. In: K.M. Barbour. (Editor) Nigeria in maps. Hodder and Stoughton, London.
- Areola, O. (1982) Soils. In: K.M. Barbour. (Editor) Nigeria in maps. Hodder and Stoughton, London.

The classification of the subgrade material is based on the soaked California Bearing Ratio (CBR) at a representative density. For structural purposes, when a material is classified according to CBR, it is implied that no more than 10% of the measured values for such a material will fall below the classification value. A soil survey of appropriate extent should therefore be conducted.

The characteristics of in-situ soils directly affect not only the pavement structure design, but may even dictate the type of pavement best suited for a given location. A careful evaluation of soil characteristics is a basic requirement for each individual pavement structure design.

4.2 Material Depth

The concept of 'material depth' is used to denote the depth below the finished level of the road to which soil characteristics have a significant effect on pavement behaviour. Below this depth, the strength and density of the soils are assumed to have a negligible effect on the pavement.

Table 4.1 shows typical thicknesses of pavement layers above the in-situ subgrade for the different road categories.

It is however recommended that on all road projects, sampling of material be done to a depth of 1.2 m

Table 4.1 Typical Pavement Thickness by Road Category

Road Category	Pavement Thickness (mm)*
A	1 000 - 1 200
B	800 - 1 000
C	800
D	700
* Total thickness of pavement above the roadbed	

4.3 Classification of Subgrades

This section is based closely on the SATCC Draft Code of Practice for the Rehabilitation of Road Pavements.⁽⁷⁾

Besides traffic loading, the subgrade strength is the other most important factor which governs pavement structural design.

The first step in the classification of the subgrade for the purpose of pavement design, involves the determination of uniform sections in terms of subgrade condition, based on geological and soil property assessments and other physical assessments such as the Dynamic Cone Penetrometer (DCP) test or in situ bearing tests.

Thereafter, the focus is on the classification of these sections in terms of the California Bearing Ratio (CBR), to represent realistic conditions for design. In practice, this means determining the CBR strength for the wettest moisture condition likely to occur during the design life, at the density expected to be achieved in the field.

The classification of subgrade condition in this guide is similar to RN31⁽⁶⁾ and is shown in Table 4.2.

Table 4.2 Subgrade Classification

	Subgrade Class Designation					
	S1	S2	S3	S4	S5	S6
Subgrade CBR ranges (%)	2	3 - 4	5 - 7	8 - 14	15 - 29	30+

Since the combination of density and moisture content wholly governs the CBR for a given material, it is clear that changes in moisture content will alter the effective CBR in the field, and particular effort must be taken to define the design subgrade condition.

The result of incorrect subgrade classification can have significant effects, particularly for poorer subgrade materials with CBR values of 5 percent and less. If the subgrade strength is overestimated (i.e. the support is actually weaker than assumed), there is a high likelihood of local premature failures and unsatisfactory performance. Conversely, if the subgrade strength is underestimated (i.e. the support is stronger than assumed), then the pavement structure selected will be thicker, stronger and more expensive than needed.

4.3.1 Representative Subgrade Moisture Content

The estimation of the wettest subgrade condition likely to occur, for design purposes, is the first stage in determining the design subgrade CBR. It is well known that moisture contents in subgrades are prone to variation due to natural effects, including rainfall, evaporation, and proximity of water table, as well as material type.

Any available local knowledge of the subgrade, locale, and prevailing conditions, should be drawn on first in determining the nominal design moisture content. Direct sampling should be undertaken if there is a clear understanding of how the sampled moisture content relates to the probable wettest condition to be encountered. If such specific information is not available, or it is felt necessary to supplement the available information, the following approach is suggested to estimate design moisture content.

- a) Areas where water-tables are normally high, regular flooding occurs, rainfall exceeds 250 mm per year, conditions are swampy, or other indicators suggest wet conditions occurring regularly during the life of the road leading to possible saturation:

Design moisture content should be the optimum moisture content determined from the AASHTO (Proctor) compaction test T-99 for the design moisture content.

- b) Areas where water-tables are low, rainfall is low (less than 250 mm per year), no distinct wet season occurs, or other indicators suggest that little possibility of significant wetting of the subgrade should occur.

Use the moisture content determined from the following formula based on the optimum moisture content (OMC) determined from the AASHTO (Proctor) compaction test T-99:

$$\text{DMC} = 0.67 * \text{OMC (\%)} + 0.8 \dots\dots\dots \text{Equation 4-1}$$

Where,

DMC = Design Moisture Content

OMC = The optimum moisture content from the AASHTO (Proctor) compaction test T-99, and the simple relationship was derived from a comprehensive investigation into compaction characteristics (Sammelink, 19913).

It is recommended to refer to RN 31⁽⁶⁾ for further details regarding the estimation of the subgrade moisture content, with respect to the location of the water table. For areas with high water tables, the top of the subgrade or improved subgrade must be raised above the level of the local water table to prevent it being soaked by ground water.

4.3.2 Classifying Design Subgrade Strength

The subgrade strength for design should reflect the probable lowest representative CBR likely to occur during the life of the road. The value will be influenced by both density achieved and moisture content. For practical purposes, it is important that the highest practical level of density (in terms of Maximum Dry Density, or MDD) be achieved from the subgrade upwards, in order to minimise subsequent deformations due to further densification under the traffic loading.

If insufficient compaction is achieved during construction, then the longer term performance of the road is likely to be negatively affected, so it is critical to ensure that good compaction is attained. It is also critical to ensure that the subgrade has been compacted to a reasonable depth in order to avoid the possibility of the road deforming, due to weakness of the deeper underlying material.

The following guidance (Table 4.3) is suggested for determining subgrade CBR for minimum subgrade compaction requirements, and for a control check on subgrade compaction during construction.

Table 4.3 Method for Classifying Subgrade Design CBR

Expected subgrade conditions	Sample conditions for CBR testing*
Saturation is likely at some periods (high rainfall areas, distinct wet season, low-lying areas, flooding, high water-table, etc)	Specimens compacted at OMC (AASHTO T99), to 100%** MOD. CBR measured after 4 days soaking***.
Saturation unlikely, but wet conditions will occur periodically (high rainfall areas, distinct wet season, water-table fluctuates, etc)	Specimens compacted at OMC (AASHTO T99), to 100%** MOD. CBR measured at OMC.
Dry conditions (low rainfall areas, water-table low)	Specimens compacted at OMC (AASHTO T99), to 100%** MOD. CBR measured with no soaking***.
Notes: * A minimum of six (6) representative samples per uniform section would be expected for classification purposes ** See (a) below regarding the use of other test moisture content/density requirements *** Cohesive materials with Plasticity Indexes (PIs) greater than 20 should be stored sealed for 24 hours before testing to allow excess pore pressures to dissipate	

4.3.3 Minimum subgrade compaction requirements

The method for classification in Table 4.3 assumes that a minimum field compaction density of 100 percent Proctor MDD (or 95 percent modified AASHTO MDD) will be attained. In most cases, with current compaction equipment this minimum should be readily achieved.

Where there is evidence that higher densities can be realistically attained in construction (from field measurements on similar materials, from established information, or from any other source), a higher density should be specified by the Engineer. The higher density should also be used in the CBR classification in Table 4.3 in place of the 100 per cent MDD value.

There may be cases where, because of high field compaction moisture contents (higher than OMC), material deficiencies or other problems, the CBR sample conditions are not realistic. In such cases, the Engineer must specify a lower target density and/or higher moisture content to be substituted for the sample conditions in Table 4.3 to represent probable field conditions more realistically.

4.3.4 Specifying the design subgrade class

The CBR results obtained in accordance with Table 4.3 are used to determine which subgrade class should be specified for design purposes, from Table 4.2.

In some cases, variation in results may make selection unclear. In such cases, it is recommended that firstly, the laboratory test process is checked to ensure uniformity (to minimise inherent variation arising from, for example, inconsistent drying out of specimens). Secondly, more samples should be tested to build up a more reliable basis for selection.

Plotting these results as a cumulative distribution curve (S-curve), in which the y-axis is the percentage of samples, less than a given CBR value (x-axis), provides a method of determining a design CBR value. This is illustrated in Figure 4.1, from which it is clear that the design CBR class is realistically S2, or 3 - 4 percent CBR. Choice of class S3 (5 - 7 per cent CBR) would be unjustified as the Figure indicates that between roughly 20 to 90 per cent of the sampled CBRs would be less than the class limits.

A good rule of thumb is to use the 10 percent cumulative percentage (percentile) as a guide to the subgrade class, on the basis that only 10 percent of the actual values would be expected to have a lower CBR than the indicated CBR. In this case, the 10 percent rule indicates a CBR of approximately 4,5 percent, thus confirming that the subgrade class of S2 is more appropriate than S3.

4.3.5 Control check on subgrade strength uniformity during construction

It is critical that the nominal subgrade strength is available to a reasonable depth in order that the pavement structure performs satisfactorily. A general rule is that the total thickness of new pavement layers (derived from the catalogue), plus the depth of subgrade, which must be to the design subgrade strength should be 800 to 1 000 mm. Table 4.4 gives recommended thickness of subgrade, with uniform strength characteristics below the designed pavement layers, for each of the subgrade classes identified in Table 4.2.

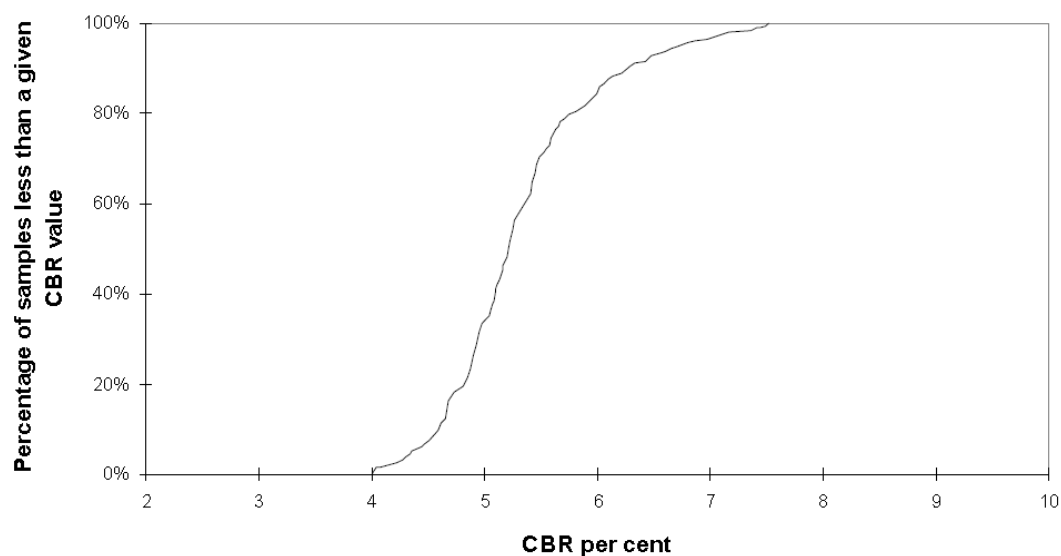


Figure 4.1 Illustration of CBR Strength Cumulative Distribution

Table 4.4 Recommended Sub-grade Thickness Below Pavement Layers

	Subgrade Class Designation					
	S1	S2	S3	S4	S5	S6
Minimum Thickness (mm)	250	250	350	450	550	650

NB: It is suggested that for Nigerian conditions, it may be more appropriate to use a minimum depth of uniformity for subgrade equal to 450 mm, even for S1, S2 and S3 subgrades.

For the stronger subgrades, especially (class S4 and higher, CBR 8 - 14 percent and more), the depth check is to ensure that there is no underlying weaker material which would lead to detrimental performance.

It is strongly recommended that the Dynamic Cone Penetrometer (DCP) be used during construction to monitor the uniformity of subgrade support to the recommended minimum depth.

APPENDIX B:- Summarises the contents of research reports obtained from the Nigerian Building and Road Research Institute which attempt to classify the subgrades of the various regions in Nigeria.⁽⁸⁾⁽⁹⁾⁽¹⁰⁾

5 Problem Soils

5.1 General

Problem soils are in-situ subgrade materials that do not meet minimum strength requirements for a subgrade, or which possess other unfavourable properties. These require special treatment before a pavement may be constructed on them. After treatment, they are reclassified into the standard subgrade classes and an appropriate pavement design undertaken.

5.2 Low-Strength Soils

Soils with a $CBR_{soaked} < 3\%$ (2% in dry climatic zones), occurring within the material depth/ design depth are described as low-strength soils. These soils require special treatment before construction of the pavement layers.

The following techniques may be employed to deal with low-strength soils:

- Treatment with lime or any other cementitious material (typically 2 to 5 percent by weight), will normally enhance CBR. Carbonation can cause longer term reversion to the original properties, so some caution and special construction measures should be adopted when using such treatment.
- Treatment with both bitumen-emulsion (typically 1,0 to 1,8 percent residual bitumen by weight) and cement (typically 1,0 – 1,5 percent by weight), will normally enhance compactibility and strength/CBR.
- Removal and replacement of soils
- Raising of vertical alignment to increase soil cover, and therefore redefine the design depth

5.3 Cohesionless Materials, Sands

Techniques which have been found to be effective in certain cases include:

- Treatment with bitumen
 - Treatment with foamed bitumen
- } stability may be a problem unless well confined

5.4 Dense Clays / Expansive Materials

Expansive soils are those that exhibit particularly large volumetric changes (swell and shrinkage), following variations in their in-service moisture contents.

The designer is referred to the following publications for a method to predict heave in clayey soils:

Technical recommendations for Highways (TRH9): Construction of Road Embankments.⁽¹¹⁾ The method is after Van der Merwe.⁽¹²⁾

Note: Expansive soils should be assessed even when they occur below design depth.

Chosen measures to minimise or eliminate the effect of expansive soils shall be economically realistic and proportionate to the risks of potential pavement damage and increased maintenance costs.

A soil is considered to be potentially expansive and requires extended investigation if:

- Field visits confirm expansiveness
- PI_w is greater than 20%

Where,

$$PI_w = PI (\% \text{ Passing the } 425\text{mm}) / 100 \dots\dots\dots \text{Equation 5-1}$$

Techniques which have been found to be effective in dealing with expansive soils in certain cases are:

- Replacement of expansive soils. Expansive soils shall be removed up to 0.6 m to 1.2 m, and back filled with fill materials meeting the general requirements for fill.
- Treatment with lime - can increase Plastic Limit (PL) and make material friable/more stable; will normally enhance CBR.
- Provide horizontal and vertical cut-off membranes above and next to expansive soils to stabilise moisture variation.

In addition to the above, the following should be noted:

- Moisture - The roadbed of expansive soil shall be kept moist and be covered with earthworks fill without undue delay
- Compaction - Attempts to densify expansive soils by processing and compaction are not necessary

5.5 Collapsible Sands and Soils

Collapsible soils can be defined as dominantly sandy materials possessing *in-situ* dry densities of less than 85 percent modified AASHTO, and oedometer-measured collapse potentials under future service stresses of greater than one percent⁽⁶⁾.

Problems associated with these materials are probably less widespread than those associated with expansive materials. As with the latter however, the important step is to recognize the occurrence of the problem. Soil engineering mapping carried out by qualified and experienced people should delineate the affected areas. Generally, the problem can be satisfactorily dealt with during the construction programme since the collapse phenomenon is not strongly time-dependent. Collapse occurs due to wetting of the soil, either with or without the addition of a load. The treatment is to induce the collapse before the placing of the embankment, and this is done by a process of compacting the *in-situ* material. At present, this is done by wetting, rolling and observing the result, rather than by specifying any particular end-result criteria. With the current development of vibrating and impact rollers, it seems likely that this approach is sufficient and that satisfactory results will be obtained by specifying a certain number of roller passes after some preliminary field trials.

This problem has recently been considered by Weston⁽¹³⁾, who has recommended compaction of the upper 0.5 m of roadbed to 90 percent modified AASHTO and of the next 0.5 m to 85 percent.

Failures of embankments due to collapse of the compacted embankment soil itself can probably be prevented by compacting the embankment soil to more than both 85 percent modified AASHTO and 1 650 kg/m³, at a moisture content not less than Proctor optimum, which should be maintained at all points in the embankment, throughout the period when the load is being increased.⁽¹⁴⁾

5.6 Dispersive Soils

Dispersive soils are clays that behave as single grained, very fine particles, rather than as a cohesive mass as clay is expected to perform. As single-grained with very

fine particles, these soils have almost no resistance to erosion, are susceptible to pipe developments in earthworks, crack easily and have low shear strength. Their excessively erodible nature is the major problem associated with dispersive soils for road construction.

A combination of simple indicator tests, observations of erosion patterns in the field, soil colour, terrain features and vegetation will together give sufficient indication that dispersive soils are present, and shall prompt precautions in design and construction of road projects. Dispersive soils cannot be identified by gradation and Atterburg Limit tests only.

The following techniques are used to counter dispersive soils:

- Particular attention to erosion protection of cut slopes and in drainage channels
- Modification with 2% to 3% lime if their use is unavoidable

5.7 Black Cotton Soils of Nigeria and Related Pavement Design

This section summarises the contents of a research report obtained from the Nigerian Building and Road Research Institute on **The Engineering Properties of Black Cotton Soils of Nigeria and Related Pavement Design**.⁽¹⁵⁾

5.7.1 Origin and Distribution

The black cotton soils are dark coloured expansive clays, characterised by the phenomena of swelling on absorption of water and shrinkage on drying. These characteristics make them highly problematic as foundations for both building and road structures.

In general, the black cotton soils derive their origin from basic igneous rocks such as basalts, rich in feldspars and mafic minerals such as montmorillonites. In Nigeria, these soils are found predominantly in the North-Eastern region of the country, lying within the Chad Basin and partly within the Benue trough. It is believed that these soils derive their origin in Nigeria, from basalts of the upper Benue trough which cover several hundred square kilometres, extending North and East of the Jos Plateau, and from quarternary sediments of lacustrine origin, from the Chad Basin consisting mainly of shales, clays and sandy sediments.

Black cotton soils generally occur in poorly-drained areas, with alternating wet and dry seasons, with an annual rainfall generally less than 1 200 mm. Such physiographical features and climatic conditions are typified in the Chad Basin, where sediments were deposited as the old lake expanded and shrank during the wet and dry periods. Other conditions conducive to their formation include the cumulative effects of leaching, alkaline environment and retention of calcium and magnesium in the soil.

5.7.2 Clay Mineralogy and Swelling Mechanism

It is considered necessary for a road engineer to have at least a basic understanding of the mechanism responsible for the swelling and shrinkage phenomena in Black Cotton soils, so that the proper precautions are taken at the design stage. This mechanism of swelling and shrinkage can best be explained by the mineralogical and chemical structure of black cotton soils.

Of the various clay minerals, viz., kaolinite, halloysite, montmorillonite and illite, it is the montmorillonite mineral which is most common in expansive clays or the black cotton soils. The basic unit for a montmorillonite consists of a gibbsite sheet between two silica sheets. The gibbsite sheet may include atoms of aluminium, iron, magnesium or a combination of these. The silicon atom in a silica sheet may interchange with say, aluminium atom of the gibbsite sheet. These structural changes termed as isomorphous changes, result in a net negative charge on the clay mineral.

The cations in water like sodium, potassium, or calcium are attracted to the negatively charged clay plates, and therefore are in a continuous state of interchange. The bond between the montmorillonite units is relatively weak, thus making it possible for water to penetrate in-between these units and cause their separation. Such a separation of the plates results in 'swelling' of the clay mass.

5.7.3 Categorisation of Black Cotton Soils

It is apparent from laboratory test data that all Black Cotton soils are not the same. The variations in their particle size distribution, clay and silt contents, liquid and plastic limits and swell potential are so wide, that the Black Cotton soil cannot be considered as just one type of soil. It is therefore necessary to develop design

specifications for each category or group or type of Black Cotton soil encountered in a particular area.

Considering the simplicity as well as efficacy, the Plasticity Index and the Free Swell are considered the most significant yet simple-to-determine parameters indicative of the swell potential of Black Cotton Soil.

The following three categories of Black Cotton Soils are proposed by the NIBRRI (Research Paper No. 1)⁽¹⁵⁾

Category I Low Swell Potential

- PI < 20
- Free Swell < 50%
- Percent smaller than 1 micron < 20

Category II Medium Swell Potential

- PI: 15 - 30
- Free Swell: 50 - 80%
- Percent smaller than 1 micron: 20 - 30

Category III High Swell Potential

- PI > 30
- Free Swell > 80%
- Percent smaller than 1 micron > 20

The above is shown diagrammatically on Figure 5.1:

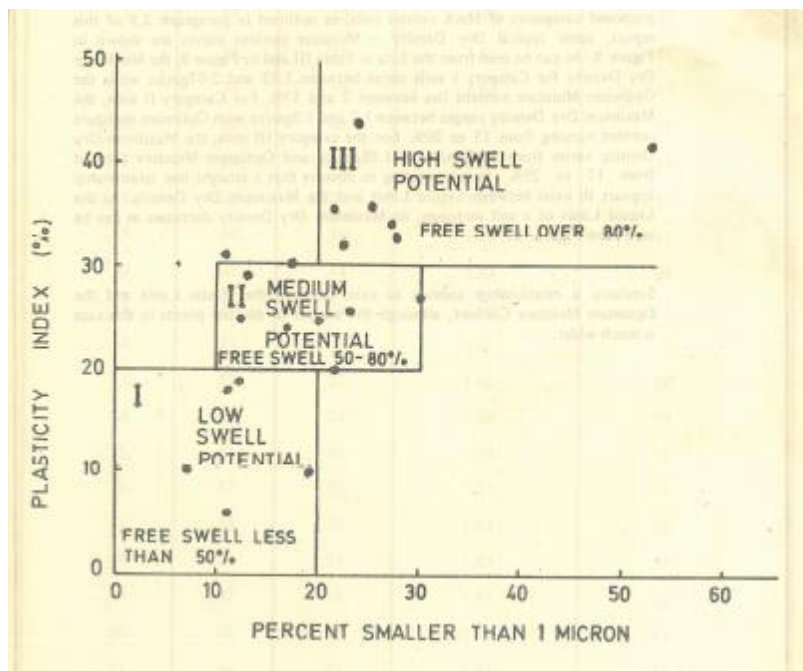


Figure 5.1 Proposed categorisation for Black Cotton Soils of Nigeria

Figure 5.2 shows the typical dry density – moisture content curves for the three categories of BC Soils

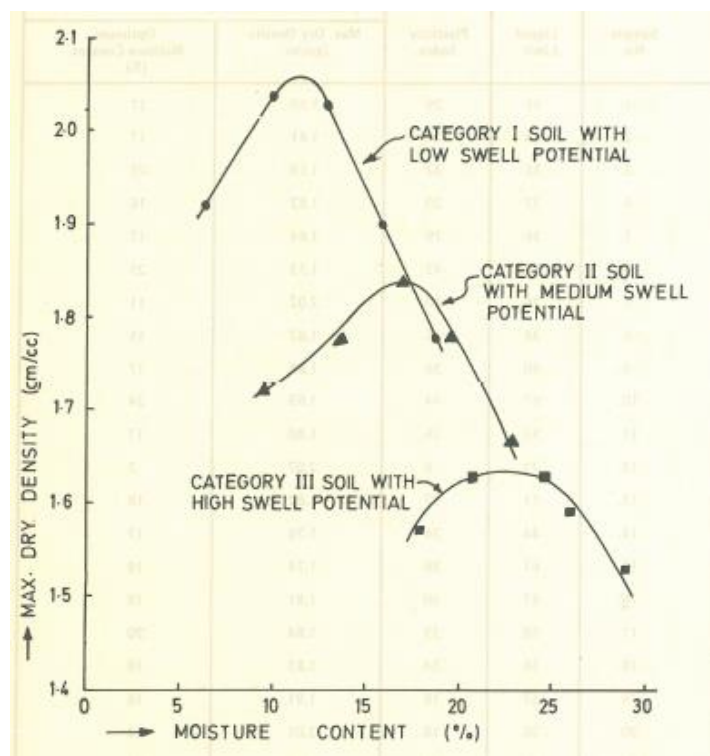


Figure 5.2 Typical Dry Density – Moisture Content Curves for the Three Proposed Categories of BC Soils

Average CBR values for each of the three categories of Black Cotton Soils are tabulated in Table 5.1

Table 5.1 Strength (CBR) Test Data

Average Test Values for Each Soil Category	Black Cotton Soil Category		
	I	II	III
Soaked CBR Value (4 day Soaking)	5	3	2
Swell during soaking in CBR mould	2.5%	3.2%	8.3%
CBR (Unsoaked) at Optimum Moisture and Maximum Dry Density	13	10	8
CBR (Unsoaked) of soil stabilised with local sand (1:1) at Optimum Moisture and Maximum Dry Density	21	16	12
Soaked CBR Value (4 day Soaking) of soil stabilised with local sand (1:1)	11	6	4

5.7.4 Pavement Design Aspects

Alternative Design Strategies

The problem peculiar to the design of road pavements on Black Cotton Soil subgrades arises from the swelling and shrinkage characteristics of these soils with changes in moisture content. Three broad strategies are available to the designer:

- i. Total replacement or partial improvement of the subgrade through stabilisation with lime, lime+cement, or mechanical stabilisation
- ii. Thicker pavement crust to counteract upward swell
- iii. Protection from moisture variations

5.8 Problem Laterites of Nigeria

The most common materials used for road construction are lateritic soils because they occur naturally with intense weathering. Lateritic soils are found in the tropical environment, where there is an intense chemical weathering and leaching of soluble minerals. Laterites are reddish brown, well-graded and sometimes extend to a depth of several tens of metres.

Problem laterite soils are those that do not yield reproducible results using standard laboratory testing procedures. The soils are difficult to evaluate as engineering construction materials. The peculiar problems of these soils have been identified as thermal and mechanical instabilities i.e. the susceptibility to significant changes on the addition of small levels of thermal or mechanical energy.

Several researchers in Nigeria have undertaken studies on dealing with both black cotton soils and problem laterites. In future updates of this volume, effort should be made to consolidate this body of knowledge into design guidelines for use in Nigeria.

6 Pavement Materials

This chapter defines the physical properties for materials to be used in the pavement structure and forms an essential part of the design of a pavement. Within the limitations given in this chapter, materials used in the structural layers of the pavement shall be selected according to criteria of availability, economic factors, and previous experience.

6.1 Unbound Pavement Materials

Generally, these materials show stress-dependent behaviour and under repeated stresses, deformation can occur through shear and/ or densification.

6.1.1 Granular Base

A wide range of materials can be used for unbound bases. These include crushed rock or stone, naturally occurring as 'dug' gravels, and various combinations of crushing and screening, mechanical stabilization (modification) or other modification. Their suitability for use depends primarily on the design traffic class of the pavement and climate, but all base materials must have a particle size distribution and particle shape which provide high mechanical stability. In particular, they should contain sufficient fines (material passing the 0.425 mm sieve) to produce a dense material when compacted.

In circumstances where several types of base are suitable, the final choice should take into account the expected level of future maintenance, and the total cost over the expected life of the pavement. The use of locally available materials is encouraged, particularly at low traffic volumes (i.e. categories T1 and T2).

In selecting and using natural gravels, their inherent variability must be taken into account in the selection process. This normally requires reasonably comprehensive characterisation testing to determine representative properties, and it is recommended that a statistical approach be applied in interpreting test results. For lightly trafficked roads, the specification requirements may be too stringent and reference should be made to specific case studies, preferably for roads under similar conditions, in deciding on suitability of materials which do not fully comply with specification requirements.

(a) Graded Crushed Stone.

Graded crushed stone can be derived from crushing fresh, quarried rock (used either as an all-in product, usually termed a crusher-run, or by screening and recombining to produce a desired particle size distribution), or from crushing and screening natural granular material, rocks or boulders, to which may be added a proportion of natural fine aggregate. After crushing, the material should be angular, but not excessively flaky in order to promote good interlock and performance. If the amount of fine aggregate produced during crushing is insufficient, additional non-plastic sand may be used to make up the deficiency.

In constructing a crushed stone base, the aim should be to achieve maximum density and high stability under traffic. Aggregate durability is normally assessed by standard crushing tests but these are not as discriminating as durability mill testing, which is the preferred method. The material is usually kept damp during transport and laying to reduce the likelihood of particle segregation. These materials are commonly dumped and spread by grader, rather than the more expensive option of using a paver, which demands greater construction skill to ensure that the completed surface is smooth with a tight finish. The Engineer should pay particular attention to this aspect to guarantee best performance. When properly constructed, however, crushed stone bases will have CBR values well in excess of 100 per cent¹.

(b) Naturally-Occurring Granular Materials

A wide range of materials including lateritic, calcareous and quartzitic gravels, river gravels and other transported gravels or granular materials resulting from the weathering of rocks, have been used successfully for bases.

The over-riding requirement for the use of such materials is the achievement of the minimum design soaked CBR of 80 percent at the specified in-situ density and moisture content conditions, and the maintaining of this strength in service (long-term durability) without undesirable volume changes in the material. Some further discussion is given below, under the sub-section on potential problem materials.

¹The CBR classification is used in this document as being the most widely adopted regional method for assessing unbound materials. Where other methods are used (such as the Texas Triaxial test), guidance may be needed on correlation for local materials. As a rule-of-thumb, however, local materials already regarded as "base" or "sub-base" quality based on previous usage and performance ought to comply with the nominal CBR requirements in this document. The main criterion is then to ensure that a satisfactory degree of compaction is achieved in the field to minimise traffic-induced consolidation and premature rutting/failure.

Guidance on material gradings which ought to meet the performance requirements is given in the form of grading limits in the specification, for various nominal maximum aggregate sizes. It must be noted that all grading analyses should be done on materials that have been compacted, since some material breakdown may occur during the process. It should also be clearly understood that the gradings are for guidance and not compliance: material outside the grading limits which is deemed to meet the CBR strength and the long-term durability requirements should be deemed acceptable. In other words, the performance criteria are the critical parameter in selecting materials.

Where the required performance cannot be consistently achieved by a particular as-dug material, mixing of materials from different sources is permissible in order to achieve the required properties, which might include adding fine or coarse materials or combinations of the two. Where blending of different materials is necessary, it has been found that a high proportion of coarser particles (more than 10 mm diameter) should have angular, irregular or crushed faces, since this aids in particle interlock and stability. By the same token, the amount of smooth, rounded, aggregate particles should be kept as low as possible, and preferably not more than 50 percent of the coarse particle volume.

The fines should preferably be non-plastic, but should normally never exceed a PI of 6, or a linear shrinkage of 3. If difficulties are encountered in meeting these criteria, the addition of a low percentage of hydrated lime or cement could be tried.

(i) Potential Problem Materials

Potential problem materials include weathered materials of basic igneous origin, including basalts and dolerites and others (unsound materials). The state of decomposition or metamorphic alteration can lead to rapid and premature failure with moisture ingress, which affects their long term durability even when stabilised. Identifying these materials can be difficult with normal aggregate classification tests and other methods must be used (including petrographic analysis and soundness tests such as soaking in ethylene glycol²). Where there is any doubt about a material's soundness or suitability, it is advisable to seek expert advice where local knowledge is insufficient.

² Chemical soundness tests such as sodium and magnesium sulphate tests are not regarded as such good indicators as the technique of soaking in ethylene glycol

Marginal Quality Materials:

There are many examples where gravels, which do not conform to normal specifications for bases, have been successfully used. Generally, their use should be confined to the lower traffic categories (T1 and T2), unless local evidence indicates that they could perform satisfactorily at higher levels. The engineer is advised to be duly cautious if some extrapolation of performance appears warranted, and to ensure that the basis of the good behaviour is reasonably understood. In most cases, the presence or absence of moisture will alter the in-situ behaviour of such materials, which is why the CBR is normally assessed under soaked (worst-case) conditions.

(c) Wet- and Dry-Bound Macadams

This is a traditional form of construction, regarded as comparable in performance with a graded crushed stone that has been used successfully in the tropics. Two nominal types are used: dry-bound and wet-bound. They are often constructed in a labour-intensive process whereby the large stones are arranged by hand.

The materials consist of nominal single-sized crushed stone and non-plastic fine aggregate filler (passing the 5.0 mm sieve). The fine material should preferably be well graded and consist of crushed rock fines or natural angular pit sand.

Both processes involve laying single-sized crushed stone (often of either 37.5 mm or 50 mm nominal size), in a series of layers to achieve the design thickness. Each layer of coarse aggregate should be shaped and compacted and then the fine aggregate spread onto the surface. The compacted thickness of each layer should not exceed twice the nominal stone size.

For dry-bound, the fines are vibrated into the voids to produce a dense layer. In wet-bound (water-bound macadam); the fines are rolled and washed into the surface to produce a dense material. Any loose material remaining is brushed off, and final compaction carried out usually with a heavy smooth wheeled roller.

This sequence (large stone, compaction, void filling) is then repeated until the design thickness is achieved. Production economy can be obtained if layers consisting of 50 mm nominal size stone and layers of 37.5 mm nominal size stone are both used, to allow the required total thickness to be obtained more precisely, and to make better overall use of the output from the crushing plant. Aggregate

hardness, durability, particle shape and in-situ density should conform to those used for graded crushed stone.

Due to the method of construction for macadams, the finished surface may be relatively bumpy and achieving an acceptable riding quality may require an asphalt levelling course, as well as surfacing. Generally it is more economical to use a properly specified crusher-run, which will provide a better finished riding surface.

The wet-bound operation should not be even considered where water sensitive, plastic materials are used in the sub-base or subgrade, as it is practically impossible to prevent moisture ingress (or even saturation) during construction. If this method of base construction is used, it should therefore be undertaken on a stabilised sub-base which will minimize the risk of damage to underlying layers.

6.1.2 Granular Sub-base

The sub-base may fulfil several requirements apart from its load-spreading capability as part of the pavement structure, including forming a working platform for the construction of the upper pavement layers, and as a separation layer between subgrade and base. The choice of sub-base material therefore depends on the design function of the layer, as well as the anticipated moisture regime both in service and at construction.

A nominal minimum CBR of 30 percent is required at 95 percent modified AASHTO MDD (test method T-180). Where construction traffic loading or climate is severe during construction, the Engineer is advised to specify more stringent requirements. Broadly, the poorer the conditions, the lower should be the limits on PI and linear shrinkage, and the more the need for a well-graded better-quality material. Conversely, for less severe conditions, particularly in drier areas, some relaxation of these requirements may be deemed justifiable.

In wet areas or if saturation of the layer is anticipated at any time during its life (for example, if used as a drainage layer, or if water might penetrate at some stage due to poor surface maintenance and a permeable base), the CBR must be determined from samples soaked in water for four days. In drier areas, the Engineer may consider an un-soaked test, but it is strongly advised that the standard soaked test is adhered to whenever possible. This is because, even in nominally dry areas,

there may still be some likelihood of wetting or saturation of the sub-base during its life, the observed effect of which is to cause marked rapid deterioration of the road.

6.1.3 Granular Selected Layer

In a number of cases, particularly for the poorer subgrade support conditions (class S1, S2 and S3); selected layers are required to provide sufficient cover on weak subgrades (see Appendix C design catalogue).

The requirements are more relaxed than for sub-bases, with the main criterion being a minimum CBR strength of 15 percent at 93 percent of the modified AASHTO MDD (test method T-180), at the highest anticipated moisture content in service. Estimation of this moisture content must take into account the functions of the overlying sub-base layer and its expected moisture condition, and the moisture conditions in the subgrade. If either of these layers is likely to be saturated during the life of the road, then the selected layer should also be assessed in this state.

Where possible, selected materials should be homogeneous and relatively insensitive to moisture change on bearing capacity (CBR strength).

6.2 Treated Materials

This section provides guidance on the use of cemented materials as base and sub-base layers in the pavement structure. In this document, the term cemented materials covers the main categories of treatment or stabilization with Portland cement, treatment with lime, and treatment with bitumen emulsion. For more complete discussion of these materials, RN31¹ is recommended as a source for cement and lime treatments. For bitumen emulsion treatment, the Asphalt Academy of South Africa has developed guidelines for the use of these materials.⁽¹⁶⁾ The use of other materials having natural cementing action (pozzolans), such as pulverised fuel ash (PFA), is not specifically discussed here, although some of the design considerations will be similar to the materials considered here. The Engineer is advised to draw on established local practice and specialist advice if the use of pozzolans may be warranted.

An overriding consideration in the use of cemented materials is that treatments will be applied in-situ, with the main intention of enhancing the suitability for pavement construction of locally available materials, and avoiding the need to import other

materials. This can usually lead to more cost-effective use of available materials but, as noted in the guidelines, the economic viability of possible alternative approaches should be assessed prior to finalising the pavement design.

Beneficial properties that will normally be sought or attained for these types of materials, compared with the untreated parent material, include:

- Increased strength or stability
- Increased layer stiffness and load-spreading capability
- Increased resistance to erosion
- Reduced sensitivity to moisture changes
- Reduced plasticity.

Potential problems or pitfalls with these types of material, of which the Engineer should be aware in their application, include:

- Propensity to crack, through traffic loading or environmental conditions (thermal and shrinkage stresses), particularly with cement treatment.
- Degradation of the cementing action due to carbonation (carbon dioxide), specifically for cement and lime treatment.
- Requirement for greater levels of skill and control during construction (compared with untreated materials) to achieve satisfactory results.

Results from pavements using bitumen emulsion treated materials indicate that this type of material is immune to the first two potential problems, but it is more expensive and requires greater levels of skill and control during construction (compared with cement stabilised materials) to achieve satisfactory results.

Construction of satisfactory cemented layers is largely dependent on producing well-mixed homogeneous materials. This therefore means that in-situ plant mixing is recommended for the best control and results. However, this may be impractical for certain applications, and lime treatment is usually only practical by mix-in-place methods. The underlying need to produce a homogeneous mix should, nevertheless, remain the principal requirement.

6.2.1 Treatment/ Stabilisation with Portland Cement

While a range of materials can be treated with cement, the use of high cement content (say 5 percent or more) should be avoided for both economic and for

performance considerations. In particular higher cement contents can lead to greater cracking potential, which may detract from the overall performance of a pavement. For this reason it is now common practice to set both upper and lower bounds on the strength of these materials to minimize the detrimental effects of cracking, on the basis that the formation of closer-spaced, narrower cracks (which occur with lower strength material) is more desirable than wider-spaced, wide cracks (which occur for stronger cemented materials). The latter causes much greater loss of structural integrity of the layer, as well as greater susceptibility to reflection cracking through overlying layers, and the potential for undesirable moisture ingress to the pavement.

As a guide, material suitable for cement treatment will normally have a low Plasticity Index (less than 10), with a reasonably uniform grading. Materials with higher PIs can first be treated with lime (modified), prior to cement treatment. Direct treatment with cement of materials with higher PIs is unlikely to be satisfactory. Laboratory trial mixes should be made, where such treatment appears to have potential, for a range of cement contents (typically 2, 4 and 6 percent by weight), at mix moisture contents appropriate to field mixing, and to a dry density which reflects probable field compaction.

Seven days moist curing at 25°C should be allowed, where specimens are either wax-sealed or wrapped in plastic cling-film then sealed in plastic bags, and kept out of direct sunlight, to represent on site conditions. This allows the strength gain that should be achieved in practice during site curing. Strength testing, however, should be after a further four hours soaking of the specimens (again at 25°C), with specimens tested direct from the waterbath to represent worst case operational conditions. In dry regions, where the possibility of saturation of the layer is deemed negligible, it may be more realistic to allow some drying out prior to testing (say 24 hours at 25°C, kept out of direct sunlight). Strength results should be plotted against cement contents in order to determine the design cement content. A reasonably well-defined relationship between strength and cement content should be obtained, and it is advisable to plot the average strength of each set of specimens as well as the individual results to view the overall correlation. In the case that unexplainable or anomalous results obscure the picture, further testing should be undertaken.

Depending on the layer application, the design cement content should ensure that the strength from the above process should be between 0.75 and 1.5 MPa, or be

between 1.5 to 3 MPa, based on specimens of nominal height to width/diameter ratios of 1:1. Generally, this should be based on the average strength relationship and the cement content to achieve the mid-range values (i.e. target strengths of 1.1 MPa and 2.2 MPa respectively). Where specimens of height to width/diameter, ratios of 2:1 are used, the corresponding ranges should be 0.6 to 1.2 MPa and 1.2 to 2.4 MPa.

The catalogue (APPENDIX C:) indicates the specific strength range which should be used, depending on the layer application, and for some designs includes a requirement for a 3 to 5 MPa UCS. This should be determined from the same process. Corresponding strength bounds for specimens of height to width/ diameter ratios of 2:1 are 2.4 to 4 MPa respectively.

Long-term durability of the material will normally be satisfactory if the parent material is sound. It should be checked, however, if any doubt at all exists about the mixture and a wet-dry brushing test has been found to be a suitable method.

6.2.2 Treatment with Lime

Addition of lime has been found very effective on many materials with high PIs, normally greater than 10, which will not respond so well to cement treatment. It may be used in order to lower the PI of materials otherwise within specification limits, as a pre-treatment (for the same purpose) of materials that might then be treated with cement or bitumen emulsion, to produce a suitable road building material, or as a strengthening agent like cement.

The quality control of the lime products can differ considerably, so the engineer must firstly confirm that both production rate and quality are satisfactory for the need identified. Two main categories of lime can be produced: hydrated and un-hydrated (quick) lime. Use of quicklime is strongly cautioned against due to health risks, and its use for road building is already banned in a number of countries.

Compared with cement, the strength and stiffness gains are less marked and the cementitious reaction is slower, so that (depending on the parent material) measurable changes can take place over a number of years. By the same token, the initial effect of lime addition, particularly to wet soils, is rapid and the chemical reaction leads to increases in strength and trafficability of such materials.

Lime treatment can be used for both base and sub-base construction, adopting the same strength limits for cemented material (as given above), and there are many examples of its successful use throughout the African continent. In selecting design lime content for sub-base usage, the same procedure used for Portland cement addition as outlined above should be followed, with the major difference in the curing time allowed. For lime, this should be 11 days moist curing instead of seven days. Testing should then be conducted after a further four hours soaking, as indicated for the cemented material. It should be noted that for strength control during construction, the curing regime above is impractical, and the Engineer should determine seven day minimum strength limits for this purpose.

6.2.3 Treatment with Bitumen Emulsion

This section will be expanded upon in Chapter 7.

6.3 Bituminous Materials

For this discussion, the term 'bituminous materials' covers asphalt base and surfacing materials, and surface dressings. This section is intended to highlight some of the more important considerations in their application, without going into specific detail, because it is assumed that such materials will already form part of established road construction techniques. More complete details of these types of materials can be found in RN31¹ or other guides.

Prime and tack coats are not specifically discussed here, but their correct use is implicitly assumed in bituminous layer applications. The use of tar as a binder is not specifically excluded in the following discussion, but its use is not encouraged due to acknowledged health hazards as a cancer-causing agent. It is strongly urged that all endeavour to phase out the use of tar and substitute an oil-based bituminous binder.

6.3.1 Asphalt Pre-Mix Base and Surfacing

Asphalt premixes are plant-produced bituminous mixes using good quality aggregates, hot mixed, transported to the site, and laid and compacted while still hot. Minimum practical thicknesses, depending on the aggregate size, can be as low as 25 mm or so. For the designs in this guide, the minimum asphalt premix

surfacing thickness is 40 mm. The mixes must be designed to provide adequate deformation (rutting) resistance, adequate fatigue resistance, good load spreading (high stiffness), and good durability while being sufficiently workable during construction to allow satisfactory compaction.

In particular, the load spreading/deformation resistance requirements (necessitating a high stiffness) can conflict with the need for fatigue resistance (usually necessitating more flexibility). Thus the design of suitable asphalt premixes should be regarded as a specialist function, whereby the asphalt producer should be given a performance-related specification to meet, using his particular expertise to ensure mix compliance.

Commonly used bituminous premixes include asphalt concretes, bitumen macadams, rolled asphalts, and mastic asphalts. These have been developed over the years from different backgrounds, essentially to make use of local aggregates and to provide similar desirable performance characteristics, but differ in composition and design approach. Where possible, therefore the Engineer should make use of local knowledge of satisfactory performing materials, and be guided by the asphalt producer.

Primary practical considerations for asphalt premixes include:

- Bitumen content
 - Air voids
- } influencing long-term durability

Marshall stability and flow criteria influencing performance and the exact requirements will differ, depending on the application as either base or surfacing. Factors which will influence selection of specific parameter values include design trafficking level, operating temperature, incidence of overloading, channelization of traffic, and gradient/terrain. Clearly, the harsher the operating environment, particularly related to the abovementioned factors, the more stringent the specification required. The Engineer should therefore draw on specialist advice for the particular application in defining the asphalt premix specification.

6.3.2 Surface Dressing

Surface dressings (or surface treatments or seals) are produced in-situ, generally using either penetration grade bitumen, or bitumen emulsions, as the binding and sealing agent. Bitumen-rubber binder (in which natural and/or synthetic rubber from old vehicle tyres mainly is blended with a bitumen binder) has also been used successfully to provide a resilient, durable, binding agent with greater resistance to deformations and cracking. Its use may be appropriate on more heavily trafficked roads where vehicle overloading is significant, or where there are high deflections measured on the pavement surface.

Hard, durable, single-sized aggregate chippings are normally used to provide a non-skid running surface. More recently, graded aggregate seals (Otta seals) have been shown to be highly successful under light traffic, and result in more cost-effective use of material with a more "forgiving" construction requirement.⁽¹⁷⁾

Bitumen binders (penetration grades, cutbacks, bitumen-rubbers and polymer modified binders) are normally applied hot, and emulsions may be applied cold, although low water content emulsions (sometimes used on more heavily trafficked roads) can also be gently heated to aid application. The underlying requirement is that the binder on application should be sufficiently fluid to spread evenly and have good adhesion with the stones. The other requirement, particularly for remedial sealing, is for the binder to then revert to its harder, stiffer (ambient condition) viscosity within a reasonable time, so that trafficking can start as soon as possible.

It is generally advised to use cutback bitumen, of medium to rapid curing, as this will normally satisfactorily fulfil the requirements indicated above.. It should be noted that it is not advisable to use cutback bitumen under hot ambient conditions. The Engineer should in any case, draw on established local practice for the particular conditions of application.

There are a number of different variations of surface dressings, with single surface treatments (or spray-and-chip) being the cheapest and simplest, ranging through double seals and more sophisticated treatments such as slurry and Cape seals. The Cape seal is a combination of a surface dressing with a slurry seal on top which has been found to be effective where a surface dressing alone may deteriorate too quickly under heavier trafficking. Single surface treatments can be extremely effective when used to reseal existing surfaced pavements, while double surface

treatments should be used on new construction. Where traffic loading conditions are particularly severe, the use of a bitumen-rubber premix with a single surface treatment has been found particularly effective and long-lived.

Common characteristics of all properly constructed new surface dressings are their ability to keep out moisture, together with their inability to rectify inherent riding quality/ roughness deficiencies from the underlying layer. In other words, surface dressings cannot be used to remedy riding quality problems.

Practical considerations in the use of surface dressings include:

- Aggregates must be clean
- Aggregates must be sufficiently strong and durable
- Aggregates must bond with the selected binder. Use of pre-coating may assist the bonding process
- Binders must be applied uniformly to the specified application rate
- Stones must be well shaped (not flaky or elongated) and nominally single-sized
- Rubber-tyred rollers are preferred for good stone embedment without crushing

The Engineer is advised to use a seal design guideline for detailed guidance on all aspects of seal selection, design and construction including:

- Factors influencing the performance of surfacing seals
- Pre-design investigations
- Selection of appropriate surfacings
- Criteria for determination of the choice of binder
- Surface preparation/pre-treatment
- Design and construction of seals
- Recommended material specification, as well as process and acceptance control, maintenance planning and budgeting, construction of seal work using labour-intensive methods, life expectancy of seals, relative cost of surfacings, selection of type of reseal and stone spread rates.

Surface dressings will deteriorate under both the effects of trafficking and time (aging of the binder), and should be expected to require remedial action within the design life of the road. Deterioration will normally take the form of loss of the

sealing ability through cracking, and/or the loss of texture through stone loss or smoothing as stone gets pushed in. Normal remedial action would be application of a new seal, as part of a periodic maintenance programme, and this should be considered a standard requirement which should be taken into account when selecting the pavement structure. Failure to maintain surface dressings is likely, therefore, to lead to a reduced pavement life.

6.4 Materials Strength Characteristics

The designs given in this guide are based on the nominal material strength classifications given in Table 6.1. For structural purposes, this provides a guide to the probable performance, assuming that no unexpected deterioration (for example, due to water ingress) takes place. The full specifications, given elsewhere, include a number of other indicative properties to assure that such deterioration ought not take place during the life of the road.

For the granular materials, only a minimum strength requirement is specified since there are usually no disadvantages in attaining higher strengths, and long-term performance is likely to be better in such cases. In line with foregoing discussions, however, it should be noted that density achieved is critically important if deformation under subsequent trafficking is to be minimised.

In contrast to just a minimum strength requirement, distinct upper and lower strength limits are placed on cemented materials (here meaning use of a Portland cement binder), due to the propensity of strongly cemented materials to form wide, widely-spaced, cracks which can reflect through overlying layers and open the pavement to moisture ingress, as well as losing structural integrity. The strength bounds are intended to ensure that any detrimental effects from cracking of the layer, which is virtually unavoidable in this type of material, are minimised by ensuring closer-spaced narrow cracks.

Table 6.1 Nominal Strength Classification of Materials in the Design Catalogue (satcc)

Layer	Material	Nominal strength
Base	Granular	Soaked CBR>80% @ 98% mod. AASHTO density
	Cemented	7 day UCS*1.5 - 3.0 MPa @ 100% mod. AASHTO density (or 1.0 - 1.5 MPa @ 97% if modified test is followed)
	Bituminous	See specification
Sub-base	Granular	Soaked CBR>30% @ 95% mod. AASHTO density
	Cemented	7 day UCS*0.75 - 1.5 MPa @ 100% mod. AASHTO density (or 0.5 - 0.75 MPa @ 97% if modified test is followed)
Capping/ selected	Granular	Soaked CBR>15% @ 93% mod. AASHTO density
* 7 day unconfined compressive strength		

Note: Samples for UCS tests are mixed and left for two hours before being compacted into 150 mm cubes. These samples are then moist cured for seven days and soaked for seven days in accordance with BS 1924. (For further details refer to TRL, RN 31). The UCS test shall be conducted according to BS 1924: Part 2:1990.

It should be recognised at the outset, that the use of cemented layers will only normally be considered if there are not suitable granular materials available locally. The first consideration is therefore to determine what local materials could be feasibly used, and how these could meet the nominal requirements without significant processing (such as crushing, screening and recombining, or mechanical or chemical stabilization).

Bearing in mind that the cost of transport of materials becomes a major cost factor if materials must be brought in to the site from a distance. It is usually cost-effective to try to utilise the local materials, even if this would then necessitate some form of processing. As indicated above, this may take various forms, but the choice is of course, ostensibly a matter of cost and economy and in most cases the pavement designer must select materials accordingly.

In the case of certain "problem" materials (requiring some form of processing to comply with nominal specification requirements, other than crushing or screening), the following techniques might be considered in order to improve their road-building potential. No specific details are given here, however and the Engineer should determine the most appropriate method based on local experience, ad hoc trials and/or specialist advice.

7 Other Pavement Materials

7.1 Bitumen Emulsion

According to a Research and Development Occasional Paper published by FERMA (Series 1, Maiden Edition, January 2013), Bitumen Emulsion has replaced Cut-Back Bitumen, which is no longer be used in Nigeria.

Bitumen Emulsion is liquid asphalt cement emulsified in water. The emulsifying agent is sometimes called the surfactant, which is composed of large molecules. All bitumen emulsions are designed to eventually break, or revert to bitumen and water.

7.1.1 Names and Classification of Emulsions

Emulsions are classified by their ionic charges as anionic and cationic. Cationic emulsions begin with a “C”. If there is no “C” the emulsion is usually anionic. The charge is important when designing an emulsion for compatibility with certain aggregates. Cationic has more affinity with commonly used aggregates in Nigeria. Anionic is for aggregates such as limestone.

After the charge designation, the next set of letters describes how quickly an emulsion will set or coalesce to continuous asphalt mass. Asphalt emulsions are also classified according to the time it takes them to “break” or come out of the suspension, and are referred to as RS (Rapid Set), MS (Medium Set), SS (Slow Set), and QS (Quick Set).

After the classification, there is a series of numbers and letters that further describe the characteristics of the emulsions. The number 1 or 2 designates the viscosity of the emulsion, with the number 1 meaning lower viscosity or more fluid, and 2 meaning higher viscosity. If there is an “h” or “s” at the end of the name, the “h” indicates a harder asphalt base and the “s” a softer asphalt base.

For example, SS-1h is a slow-setting emulsion, with a lower viscosity made from a relatively hard base asphalt.

RS emulsions break rapidly and have little or no ability to mix with an aggregate.

MS emulsions are designed to mix with aggregates, and are often called mixing-grade emulsions. MS emulsions are used in cold recycling, cold and warm dense-graded aggregate mixes, patch mixed and other mixes.

i. SS and QS Emulsions

SS Emulsions are designed to work with fine aggregates to allow for maximum mixing time and extended workability. They are the most stable emulsions and can be used in dense-graded aggregate bases, slurry seals, soil stabilisation, asphalt surface courses and some recycling. SS emulsions can be diluted with water to reduce their viscosity so they can be used for tack coats, for seals and dust palliatives. SS emulsions are also used as driveway sealers.

QS emulsions work well with fine aggregates, but are designed to break faster than SS emulsions. QS emulsions are used in micro-resurfacing and slurry seal designs. The quick break allows for faster opening to traffic.

ii. High Float Emulsions

An “HF” that precedes the setting time designation indicates a High Float emulsion. HF emulsions are designed so the emulsifier forms a gel structure in the asphalt residue. The thicker asphalt film allows these emulsions to perform in a wider temperature range. High Floats are used in chip seals, cold mixes and road mixes.

iii. Polymers

A “P” may be added to the set designation to show the presence of polymer in the emulsion. An “L” indicates the presence of latex polymer. For example, CRS-2P is a cationic, rapid setting emulsion having a higher viscosity and containing some polymer.

Polymers and latex are used to add strength, elasticity, adhesion and durability to the pavement. Polymer asphalt emulsions can be less brittle at low temperatures, to resist cracking and stiffer at higher temperatures to resist rutting and bleeding. Polymers permit the application of micro surfacing in wheel path ruts and other locations where multiple stone depths are required.

Table 7.1 shows bitumen emulsion types, characteristics and general use.

Table 7.1 Bitumen Emulsion Types, Characteristics and General Use

Type/ Grade	Percent Asphalt (MIN)	Types – Percent Cutback	Preparation (Min-Max)	General Uses
SS-1	57	Water 43	100-200	Tack
SS-1H	57	Water 43	40-90	Tack, Slurry Surface Treatment
CSS-1	57	Water 43	100-250	Tack
CSS-1H	57	Water 43	40-90	Tack, Slurry Surface Treatment
RS-1	55	Water 45	100-200	Bituminous Seal Coat
RS-2	63	Water 37	100-200	Bituminous Seal Coat
CRS-1	60	Water 40	100-250	Bituminous Seal Coat
CRS-2	65	Water 35	100-250	Bituminous Seal Coat

7.2 Bitumen Stabilised Materials

Considerable research has gone into bitumen stabilised materials over the last decade. The experience in South Africa and Australia, and the guideline documents published in this regard are considered to represent global best practice, and offer a good reference for the use of these materials in Nigeria.

The user of this manual is referred to the following document (freely available on the internet):

Technical Guideline: Bitumen Stabilised Materials. *A guideline for the Design and Construction of Bitumen Emulsion and Foamed Bitumen Stabilised Materials.*

TG2. Second Edition. May 2009

Asphalt Academy

<http://www.asphaltacademy.co.za/Documents>

7.2.1 Introduction to Bitumen Stabilised Materials⁽¹⁶⁾

The following brief extract is obtained from the above document as an introduction to Bitumen Stabilised Materials:

Bitumen Stabilised Materials are pavement materials that are treated with either bitumen emulsion or foamed bitumen. The materials treated are normally granular materials, previously cement-treated materials or reclaimed asphalt (RA) layers. Where an existing pavement is recycled, old seals or asphalt surfacing is usually mixed with the underlying layer and treated to form a new base or sub-base layer.

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

The addition of bitumen emulsion or foamed bitumen to produce a BSM results in an increase in material strength and a reduction in moisture susceptibility as a result

of the manner in which the bitumen is dispersed among the finer aggregate particles.

BSM – Emulsion

With BSM-emulsion the bitumen emulsion disperses preferentially amongst the finer particles, but not exclusively. There is some “painting” of the larger particles by the bitumen emulsion. This is illustrated schematically in Figure 7.1.

With bitumen emulsions, there is a chemical bond between the bitumen and the aggregate promoted by the emulsifier

BSM - Foam

Foamed bitumen distributes exclusively to the finer particles, producing “spot-welds” of a mastic of bitumen droplets and fines. This is illustrated in Figure 7.1.

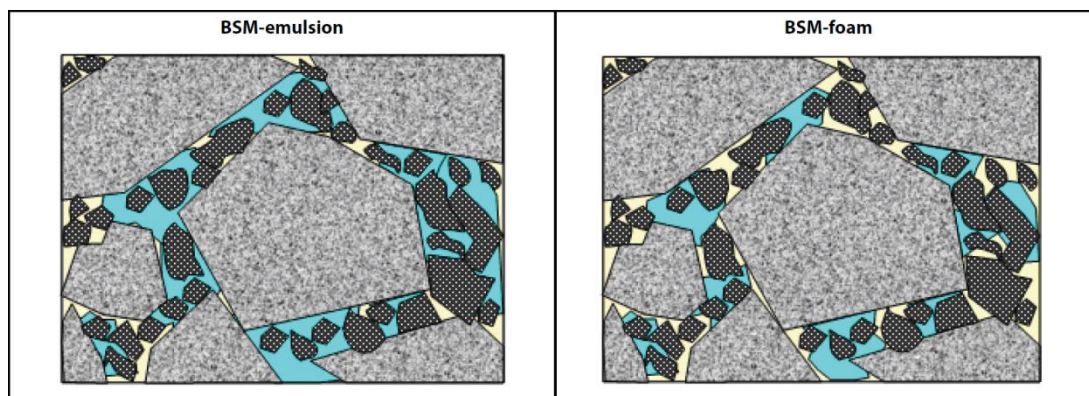


Figure 7.1 Aggregate and Binder Bond for BSMs

Such “non-continuous” binding of the individual aggregate particles makes BSMs different from all other pavement materials. The dispersed bitumen changes the shear properties of the material by significantly increasing the cohesion value, whilst effecting little change to the internal angle of friction. A compacted layer of BSM will have a void content similar to that of a granular layer, not asphalt. BSMs are therefore granular in nature and are treated as such during construction

7.2.2 Benefits of Bitumen Stabilisation

The primary benefits of using BSMs are:

- The increase in strength associated with bitumen treatment allows a BSM to replace alternative high-quality materials in the upper pavement.
- Improved durability and moisture sensitivity due to the finer particles being encapsulated in bitumen and thereby immobilised.
- Lower-quality aggregates can often be successfully used.

BSM – Emulsion	BSM - Foam
These materials may be used for materials with a low fines content	These mixes may be produced in bulk and stockpiled close to the point of application, to be placed and compacted at a later stage. This provides flexibility in mix manufacturing.

- The typical failure mode of a BSM (permanent deformation) implies that the pavement will require far less effort to rehabilitate when the terminal condition is reached, compared to a material that fails due to full-depth cracking
- BSMs are not temperature sensitive, unlike hot-mix asphalts. This is because the bitumen is not continuous throughout the mix.

BSM – Emulsion	BSM - Foam
Layers of BSM-emulsion may be subjected to traffic within a few hours (after the bitumen emulsion in the upper portion of the layer breaks)	BSM-foam mixes can be successfully used for treating in-situ material with a relatively high moisture content After compaction, layers of BSM-foam have sufficient strength to be trafficked immediately with little detrimental effect.

The complete Technical Guideline should be consulted before electing to utilise BSMs in a pavement.

7.3 Recycled Asphalt

Broadly speaking, asphalt recycling is a process in which asphalt reclaimed from an existing road, typically through milling of surfacing layers, is combined with new aggregate and new binder in a mixing plant to produce a recycled asphalt material. The reclaimed asphalt contains roughly 95% of high quality aggregate and 5% of aged bitumen, both valuable non-renewable resources.

While several factors influence the use of RAP in asphalt pavement, the two primary factors are economic savings and environmental benefits. RAP is a useful alternative to virgin materials because it reduces the use of virgin aggregate and the amount of virgin asphalt binder required in the production of HMA. The use of RAP also conserves energy, lowers transportation costs required to obtain quality virgin aggregate, and preserves resources. Additionally, using RAP decreases the amount of construction debris placed into landfills and does not deplete non-renewable natural resources such as virgin aggregate and asphalt binder. Ultimately, recycling asphalt creates a cycle that optimizes the use of natural resources and sustains the asphalt pavement industry.⁽¹⁸⁾

Two guiding principles of asphalt recycling are:⁽¹⁹⁾

- i. Mixtures containing RAP should meet the same requirements as mixes with all virgin materials.
- ii. Mixes containing RAP should perform equal to or better than virgin mixtures.

Reclaimed/ Recycled asphalt pavement (RAP) can be used as an aggregate in the recycling of asphalt paving mixtures in one of two ways. The first method (cold/ hot mix plant recycling) involves a process in which RAP is combined with new emulsified or foamed asphalt and a recycling or rejuvenating agent, possibly also with virgin aggregate, and mixed at a central plant or a mobile plant to produce cold mix base mixtures. The second, more common, method involves a process in which the asphalt pavement is recycled in-place (cold/ hot in-place recycling process), where the RAP is combined with new emulsified or foamed asphalt and/or a recycling or rejuvenating agent, possibly also with virgin aggregate, and mixed at the pavement site, at either partial depth or full depth, to produce a new mix end product.⁽²⁰⁾

7.3.1 Plant Mix Recycling

The use of processed RAP to produce conventional recycled hot mix (RHM) is the most common type of asphalt recycling and is now considered standard asphalt paving practice. There is abundant technical data available indicating that properly specified and produced recycled hot mix asphalt is equivalent in quality and structural performance to conventional hot mix asphalt in terms of rutting, ravelling, weathering, and fatigue cracking. Recycled hot mix asphalt mixtures also generally age more slowly, and are more resistant to the action of water than conventional hot mix asphalt.

The maximum limit for RAP content in RHM produced in conventional hot mix asphalt batch plants is widely considered to be 50 percent, limited by both the heat capacity of the plants and gaseous hydrocarbon emissions. As much as 60 to 70 percent RAP may be processed in drum mix plants. Special plants based on microwave technology have been developed to limit gaseous emissions from hot mix asphalt production using very high RAP contents (up to 100 percent RAP), but the cost of heating is much higher than that of conventional systems. This process was developed in California and has only seen limited use.

Reclaimed asphalt pavement must be processed into a granular material prior to use in hot mix applications. A typical RAP processing plant consists of a crusher, screening units, conveyors and stacker. It is desirable to produce either a coarse or a fine fraction of processed RAP to permit better control over input to the hot mix plant, and better control of the mix design. The processed RAP used in recycled hot mix asphalt should be as coarse as possible and the fines (minus 0.075 mm (No. 200 sieve)) minimized. Gentle RAP crushing (controlled crusher speed and clearance adjustment on exit gate) is recommended to minimize the fracture of coarse aggregate and excess fines generation.

Processing requirements for cold mix recycling are similar to those for recycled hot mix. Recycled asphalt pavement must be processed into a granular material prior to use in cold mix applications. A typical RAP plant consists of a crusher, screening units, conveyors, and stackers.

7.3.2 In-Place Recycling

The use of hot in-place recycling (HIPR) has developed rapidly over the past decade, although it is in use only on a limited basis. Simple heater-scarification units, heat reforming systems, and special techniques have been developed for heating, scarifying, rejuvenation, and remixing of up to 50 mm in depth of aged old asphalt pavement, to new hot mix quality overlay in one pass. The Asphalt Recycling and Reclaiming Association (USA) recognizes three basic HIPR processes:

- i. heater-scarification (multiple pass)
- ii. repaving (single pass)
- iii. Remixing.

The first two processes involve removal, rejuvenation, and replacement of the top 25 mm of the existing pavement. The remixing process involves incorporating virgin hot mix, with the recycled paving material in a pugmill and placement to a depth of 50 mm.

In the HIPR process, the surface of the pavement must be softened with heat prior to mechanical scarification. The HIPR process has evolved into a self-contained, continuous train operation that includes heating, scarifying, rejuvenator addition, mixing, and replacement.

CIPR (like hot in-place recycling [HIPR]), requires a self-contained, continuous train operation that includes ripping or scarifying, processing (screening and sizing/crushing unit), mixing of the milled RAP, and the addition of liquid rejuvenators. Special asphalt-derived products such as cationic, anionic, and polymer modified emulsions, rejuvenators and recycling agents have been developed especially for CIPR processes. These hydrocarbon materials are sometimes but not always, used to soften or lower the viscosity of the residual asphalt binder in the RAP material, so that it is compatible with the newly added binder.

7.3.3 Engineering Properties

Some of the engineering properties of RAP that are of particular interest when RAP is incorporated into new asphalt pavements include its gradation, asphalt content, and the penetration and viscosity of the asphalt binder.

Gradation: The aggregate gradation of processed RAP is somewhat finer than virgin aggregate. This is due to mechanical degradation during asphalt pavement removal and processing. Gradation requirements as per the General Specifications must be adhered to.

Asphalt Content and Properties: The asphalt content of most old pavements will comprise approximately 3 to 7 percent by weight and 10 to 20 percent by volume of the pavement. Due to oxidation aging, the asphalt cement has hardened and consequently is more viscous and has lower penetration values than the virgin asphalt cement. Depending on the amount of time the original pavement had been in service, recovered RAP binder may have penetration values from 10 to 80 and absolute viscosity values at 60°C (140°F) in a range from as low as 2,000 poises to as high 50,000 poises or greater.

7.3.4 Mix Design

The complexity of a mix design process varies with the level and type of recycling selected. Hot Mix Recycling where 15 percent or less RAP is blended with new aggregate and virgin asphalt requires little change from the mix design procedure used on the virgin mix because the added RAP is not expected to significantly alter the properties of the final mix. However, for higher RAP contents (>25%), a more comprehensive mix design process is needed. Blend charts need to be developed using the asphalt recovered from RAP and virgin asphalt or recycling agent to determine the percentage of RAP that provides the desired binder and mix properties in the final recycled pavement.⁽²¹⁾

Recycled Hot Mix

The use of processed RAP in hot mix asphalt pavements is now standard practice in most jurisdictions and is referenced in ASTM D3515.

The Asphalt Institute's manual on mix design methods for asphalt concrete provides a method to determine necessary mix design characteristics (such as stability, flow,

and air voids content) for either the Marshall or the Hveem mix design methods. The final mix design proportions for the recycled hot mix paving mixture will be determined by completing mix design testing, using standard procedures to satisfy applicable mix design criteria.⁽²²⁾

Cold Plant Mix

The specifications and design of cold plant mix recycling of asphalt pavements are referred to in ASTM D4215.

Although there are no universally accepted mix design methods for cold mix recycling, the Asphalt Institute recommends and most agencies use a variation of the Marshall mix design method. General procedures include a determination of the aggregate gradation and asphalt content of the processed RAP, determination of the percentage (if any) of new aggregate to be added, calculation of combined aggregate in recycled mix, selection of the type and grade of new asphalt, determination of the asphalt demand of the combined aggregate, estimation of the percent of new asphalt required in the mix, and adjustment of asphalt content by field mix trials.

Hot In-Place Recycling

Mix design procedures for HIPR are not as well established as those for conventional recycled hot mix.

The material properties of the existing asphalt pavement (to at least the depth of scarification) should be determined prior to construction, in order to permit any necessary adjustments to aggregate gradation to develop the required voids in mineral aggregate (VMA), and selection of the appropriate viscosity binder. This will require coring of the pavement to be recycled and laboratory testing of the recovered paving samples.

Unlike conventional recycled hot mix where the RAP is combined with a significant amount of new aggregate material (making up typically between 60 to 80 percent of the RHM), HIPR may involve up to 100 percent recycling of the existing pavement. Consequently, the extent to which the existing pavement can be improved or modified is limited by the condition and characteristics of the old mix.

The amount of rejuvenating agent that can be added through HIRP is limited by the air voids content of the existing asphalt. When the air voids content of the old asphalt mix is too low to accommodate sufficient recycling agent for proper rejuvenation or softening of the old asphalt binder without mix flushing, it may be necessary to add additional fine aggregate or to beneficiate with virgin hot mix to open up the mix or increase the air voids. The selection of the appropriate addition (either fine aggregate or virgin hot mix), and the amount to be added, are determined by Marshall or Hveem mix design methods.

The type of recycling or rejuvenating agent and the percentage to be added to the binder can be estimated using procedures outlined in ASTM methods D4552 and D4887. The recycling or rejuvenating agent, if used, should be compatible with the recycled and new asphalt binder.

Cold In-Place Recycling

The Asphalt Institute has recommended a modified Marshall mix type procedure for the design of CIPR mixes. Such a design initially involves obtaining samples of the candidate pavement to determine the gradation of the aggregate, the asphalt content, and the penetration and viscosity of the asphalt binder. Marshall specimens are prepared at various emulsion percentages, as initially determined by calculating the asphalt demand on the basis of aggregate gradation and deducting the percentage of asphalt in the RAP. The optimum asphalt content can be determined by a stability and air voids analysis, with target air voids in the 8 to 10 percent range, or the specimens may be evaluated using indirect tensile strength or resilient modulus testing.

It has recently been shown that the addition of virgin aggregates (20 to 25 percent) in the CIPR process results in less voids and, consequently, less flushing, and improved stability. The amount of recycling agent (either new asphalt or modifying oil) also has a significant effect on the behaviour of the mix, with the ideal range of recycling agent being somewhere between 2 and 3 percent by weight of dry RAP.

7.4 Geotextiles and Geogrids

7.4.1 Introduction

Engineers are continually faced with maintaining and developing pavement infrastructure with limited financial resources. Traditional pavement design and construction practices require high-quality materials for fulfillment of construction standards. In many areas of the world, quality materials are unavailable or in short supply. Due to these constraints, engineers are often forced to seek alternative designs using substandard materials, commercial construction aids, and innovative design practices. One category of commercial construction aids is geosynthetics. Geosynthetics include a large variety of products composed of polymers and are designed to enhance geotechnical and transportation projects. Geosynthetics perform at least one of five functions: separation, reinforcement, filtration, drainage, and containment.

Base reinforcement occurs when a geosynthetic is placed at the bottom or within the base to:

- i. Improve the service life and/ or
- ii. Obtain equivalent performance with a reduced structural section

The mechanisms associated with the incorporation of a geosynthetic include: lateral restraint, increased bearing capacity and/ or tension membrane.

Sub-base reinforcement occurs when a geosynthetic is placed at the sub-base/ subgrade interface to increase the workability for the construction platform over weak subgrade and provide improved support for the roadway structural section.⁽²³⁾

Geosynthetics are defined in ASTM D4439 as a planar product manufactured from a polymeric material that is used with soil, rock, earth, or other geotechnically-related material as an integral part of a civil engineering project, structure or system. The source materials used to produce geosynthetics typically includes one or more of the following polymers:

- Polypropylene
- Polyester
- Polyethylene
- Polyamide (nylon)

- PVC

AASHTO and ASTM provide specifications that address numerous properties of geosynthetics and the test methods that are used to define the properties.⁽²⁴⁾ Most of the primary properties and test methods for transportation projects fall under one of the following three categories:

- i. Physical properties
- ii. Mechanical properties
- iii. Hydraulic properties

7.4.2 Types of Geosynthetics for Highway Applications

- i. Geotextiles

Geotextiles are permeable materials comprised of fibres or yarns combined into a planar textile structure. Specifications for Geotextiles are provided in AASHTO M 288. The vast majority of geotextiles are either woven or nonwoven.

Geotextiles are used for strength, separation, drainage and filter purposes. Properties of the geotextile will change depending on the type of the application.

- ii. Geogrids

Geogrids consist of polymer mats constructed either of coated yarns or punched and stretched polymer sheets. They are commonly used for soil reinforcement. Geogrids are formed using a regular network of integrally connected elements with apertures greater than 6 mm to allow interlocking with surrounding geomaterials. Standard specifications for geogrids are provided in ASTM D 5262.

- iii. Geonets

Geonets are a netlike polymeric material manufactured of integrally connected parallel sets of ribs overlying similar sets of ribs. They are used for planar drainage of liquids or gases.

iv. Geocomposites

Geocomposites generally consist of a geonet or a cusped or dimpled polyethylene drainage core wrapped in a geotextile. These are often used as edge drains, wall drains, vertical drains and sheet drains. The drainage net/ core acts as a conduit for water and the geotextile wrap acts as a filter keeping the net/ core clean of soil particles.

v. Geomembranes

Geomembranes consist of impervious polymer sheets that are typically used to line ponds or landfills or, in some cases, encapsulate moisture sensitive swelling clays to control moisture. Various types of materials are used for geomembranes (e.g. polyvinyl chloride (PVC), high density polyethylene (HDPE), polypropylene (PP), polyester (PET). The thickness of these materials can range from 0.5 mm to 2.5 mm or more. Various seaming methods are used to seal multiple membrane panels together.

vi. Geosynthetic Clay Liners (GCL)

Geosynthetic Clay Liners are manufactured hydraulic barriers consisting of sodium bentonite clay sandwiched and bonded between two geotextiles or attached with an adhesive to a geomembrane. GCL are manufactured in continuous sheets and are installed by unrolling and overlapping the edges and ends of the panels. Overlaps self-seal when the sodium bentonite hydrates. Some transportation applications of GCLs include:

- Control of vertical or horizontal infiltration of moisture into a subgrade of expansive soil
- Sealing of berms for wetland mitigation
- Waterproofing walls and bridge abutments
- Lining rest area waste water treatment lagoons

7.4.3 Asphalt Reinforcement

A designer considering the use of Asphalt Reinforcing should consult the following guideline document:

Technical Guideline: Asphalt Reinforcement for Road Construction.

TG3. First Edition. November 2008

Asphalt Academy

<http://www.asphaltacademy.co.za/pubart.php>

The main purpose of the Guideline is to provide a synthesis of practical, state-of-the-art approaches to the use of ARI, based both on international best practice, plus regional knowledge and experience. The primary goal therefore is to contribute towards a reduction in the cost of rehabilitating and thereafter, maintaining asphalt pavement layers, leading to more sustainable road infrastructure provision in the southern African environment.

This Guideline covers the following materials and types of reinforcement:

- All types of materials for interlayers
- Interlayers placed in or under asphalt layers

7.4.4 Additional Geosynthetics References

The following documents should be reviewed for guidance on the use and design of geosynthetics:

- AASHTO M 288: Standard Specifications for Geotextiles
- ASTM D 4439: Standard Terminology for Geosynthetics
- Mechanically Stabilised Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines, FHWA-NHI-00-043
- Geosynthetic Design and Construction Guidelines Reference Manual, FHWA HI-95-038
- Montana Geotechnical Manual, Montana Department of Transport

8 Pavement Design

8.1 Flexible Pavements

A pavement, like any other engineering structure, is designed to withstand certain loads. In this case, the primary load that needs to be considered in the design is the total traffic that will be carried by the road. In Nigeria, as in many other countries, the standard axle load is 80 kN. The legally permissible axle load as per current legislation is 8.2 tonnes.

It is worth noting that the current legislation has been superseded by ECOWAS limits under a Decision of the ECOWAS Council of Ministers. The ECOWAS limits allow a load of 13 tonnes for the load-carrying axles, but have a very low front axle limit, resulting in similar total vehicle loads.

To give satisfactory service, a pavement must satisfy a number of structural criteria, including:

- The subgrade must be able to sustain traffic loading without excessive deformation; this is controlled by the vertical compressive stress or strain at formation level.
- Bituminous materials and cement-bound materials used in road-base design for long life must not develop fatigue cracks under the influence of traffic.
- In pavements containing a considerable amount of bituminous material, the internal deformation of these materials must be limited; their deformation is a function of their creep characteristics.
- The load-spreading ability of granular sub-bases and capping layers must be adequate to provide a satisfactory construction platform.

It is often valuable to consider several methods while carrying out a pavement design – within the applicable limitations of each respective method – in order to obtain a wider confidence of the design. This manual contains a description of some of these methods that would be suitable for use in Nigeria for different situations.

8.1.1 Design Catalogue

The catalogue used in this design manual is based on the fourth edition of Road Note 31. **It must be noted that the designs in this catalogue are only appropriate for roads which are required to carry up to 30 million cumulative equivalent standard axles per direction.**

The cells of the catalogue are defined by ranges of traffic (Chapter 3) and subgrade strength (Chapter 4) and all the materials are described in Chapter 5 and 6.

The charts are designed so that, whenever possible, the thickness of each lift of material is obvious. Thus, all layers less than 200 mm will normally be constructed in one layer, and all layers thicker than 300 mm will be constructed in two lifts. Occasionally, layers are of intermediate thickness and the decision on lift will depend on the construction plant available and the ease with which the density in the lower levels of the lift can be achieved. The thickness of the lift need not be identical, and it is often better to adjust the thickness according to the total required, and the maximum particle size, by using a combination of gradings from the General Specifications for Crushed Stone Gradings, Nigeria.

In charts 3, 4, and 7 where a semi-structural surface is defined, it is important that the surfacing material should be flexible and the granular road-base should be of the highest quality – preferably a crushed stone.

In traffic classes T6, T7 and T8 only granular road-bases of type GB1 or GB2 should be used. GB3 is acceptable in the lower traffic classes. For lime or cement-stabilised materials, the charts already define the layers for which the three categories of material be used.

The choice of chart will depend on a number of factors, but should be based on minimising total transport costs. Other factors that will need to be considered include:

- Likely level and timing of maintenance
- Probable behaviour of the structure
- Experience and skill of contractors and the availability of suitable plant
- Cost of different materials
- Other risk factors

The Design Catalogue is contained in APPENDIX C:.

8.1.2 Asphalt Institute Method

In the Asphalt Institute Design Method, the pavement is represented as a multi-layered elastic system. The wheel load W is assumed to be applied through the tyre as a uniform vertical pressure, p_0 , which is then spread by the different components of the pavement structure and eventually applied to the subgrade as a much lower stress p_1 . Experience, established theory and test data are then used to evaluate two specific stress-strain conditions. Thickness design charts were developed, based on criteria for maximum tensile strains at the bottom of the asphalt layer, and maximum vertical compressive strains at the top of the subgrade layer.

Design Procedure

The principle adopted in the design procedure is to determine the minimum thickness of the asphalt layer that will adequately withstand the vertical compressive strain at the surface of the subgrade, and the horizontal tensile strain at the bottom of the asphalt layer. Design charts have been prepared for a range of traffic loads.

The procedure consists of five main steps:

- Select or determine input data
- Select surface and base materials
- Determine minimum thickness required for input data
- Evaluate feasibility of staged construction and prepare stage construction plan if necessary
- Carry out economic analysis of alternative designs and select the best design

Step 1 Design Inputs

Design Inputs in this method are:

- Traffic characteristics – design traffic (Chapter 3)
- Subgrade engineering properties (Chapter 4)

Conversion of CBR to Resilient Modulus (M_r) is done as follows:

$$M_r(\text{MPa}) = 10.342 * \text{CBR} \dots \text{Equation 8-1}$$

$$M_r(\text{lb/in.}^2) = 1\,500 * \text{CBR} \dots \text{Equation 8-2}$$

(The above conversion factors should be used only for materials that can be classified under the Unified Classification System as CL, CH, ML, SC, SM and SP, or when the resilient modulus is less than 30 000 lb/in.². For materials with higher values, direct measurement is recommended).

- Subbase and Base Engineering Properties

Step 2 Surface and Base Materials

The designer is free to select either an asphalt concrete surface or an emulsified asphalt concrete surface, along with an asphalt concrete base, an emulsified asphalt base, or an untreated aggregate base and sub-base for the underlying layers. This will depend on the material that is economically available.

The Asphalt Institute recommends certain grades of asphalt cement that should be used for different temperature conditions. Selection is on the basis of its ability to coat aggregates at the given temperatures.

Mean annual temperatures in Nigeria are generally high, with temperatures ranging between 23°C - 31°C at the coast, and getting as high as 44°C inland. 60/70 pen and 40/50 pen bitumen is therefore recommended for use in Nigeria.

Step 3 Minimum Thickness Requirements

The AI Method has design charts for the following types of pavements:

- **Asphalt Concrete Surface and Emulsified Asphalt Base**

These pavements have asphalt concrete as surface material and emulsified asphalt as the base material. Three mix types of emulsified asphalt are used in this design and they are defined as:

Type I	Emulsified asphalt mixes made with processed, dense-graded aggregates
--------	---

Type II Emulsified asphalt mixes made with semi-processed, crusher-run, pit-run, or bank-run aggregate

Type III Emulsified asphalt mixes made with sandy or silty sands

Table 8.1 shows the recommended minimum thickness of asphalt concrete over types II and III emulsified asphalt bases. For pavements constructed with type I emulsified asphalt base, a surface treatment will be adequate. The depth of the emulsified asphalt base is determined as the difference between the total thickness (asphalt concrete surface and emulsified asphalt base), as obtained from the design charts and the minimum required thickness of the asphalt concrete as obtained from Table 8.1.

Table 8.1 Minimum Asphalt Thickness for Type I & II Base

Traffic Level ESALs	Type II and Type III
	(mm)
10^4	50
10^5	50
10^6	75
10^7	100
$> 10^7$	130

- **Asphalt Concrete Surface and Untreated Aggregate Base**

These pavements consist of a layer of asphalt concrete over a layer of untreated aggregate base and subbase courses. The design charts are given for different base thicknesses and are based on the quality requirements for base and subbase materials given in Table 8.2

Table 8.2 Base and Sub-base Requirements

Parameter	Requirements	
	Sub-base	Base
CBR, minimum	20	80
Liquid Limit, maximum	25	25
Plasticity Index, maximum	6	NP
Passing No. 200 Sieve, maximum	12	7

The Asphalt Institute also recommends that the base course be not less than 150 mm thick.

Table 8.3 gives the minimum recommended thicknesses for the asphalt concrete surface over the untreated aggregate base. The values depend on the design ESALs. In using the design charts, minimum thicknesses should not be extrapolated into higher traffic regions.

Table 8.3 Minimum Asphalt Thickness over Untreated Base

Traffic ESALs	Traffic Condition	Minimum Thickness of Asphalt Concrete
10^4	Lightly trafficked rural roads	75 mm
$> 10^4$ but $< 10^6$	Medium truck traffic	100 mm
$> 10^6$	Medium to Heavy truck traffic	125 mm

The design charts for both types of pavements are contained in **APPENDIX C**.

8.1.3 Mechanistic Design

In the design of flexible pavements, the pavement structure is usually considered as a multi-layered elastic system, with the material in each layer characterised by certain physical properties that may include the modulus of elasticity, the resilient modulus, and the Poisson ratio. It is usually assumed that the subgrade layer is infinite in both the horizontal and vertical directions, whereas the other layers are finite in the vertical direction and infinite in the horizontal direction. The application of a wheel load causes a stress distribution, which can be represented as shown in Figure 8.1. The maximum vertical stresses are compressive and they occur directly under the wheel load. These decrease with increase in depth from the surface.

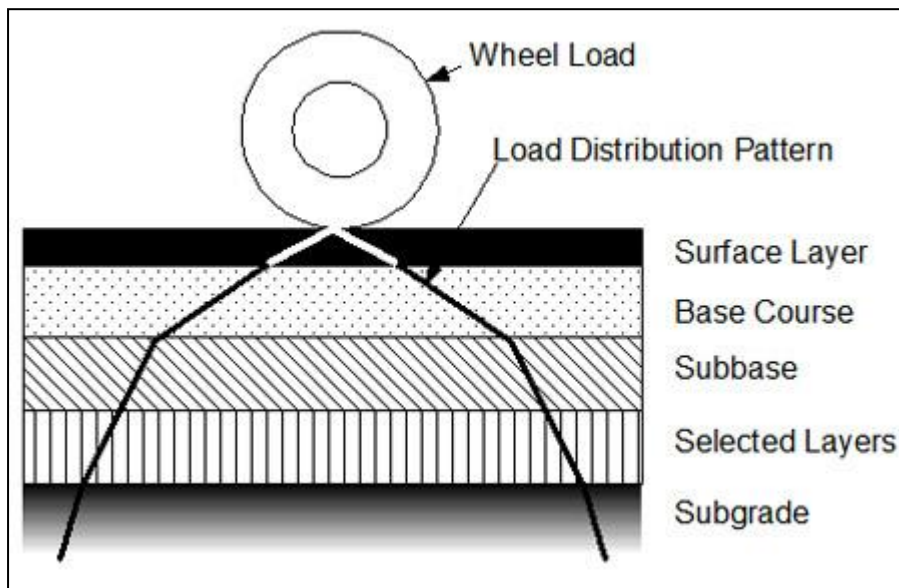


Figure 8.1 Wheel Load Distribution in Pavement Layers

The maximum horizontal stresses also occur directly under the wheel load but can be either tensile or compressive as shown in Figure 8.2. The load and pavement thickness determine whether the horizontal compressive stresses will occur above or below the neutral axis.

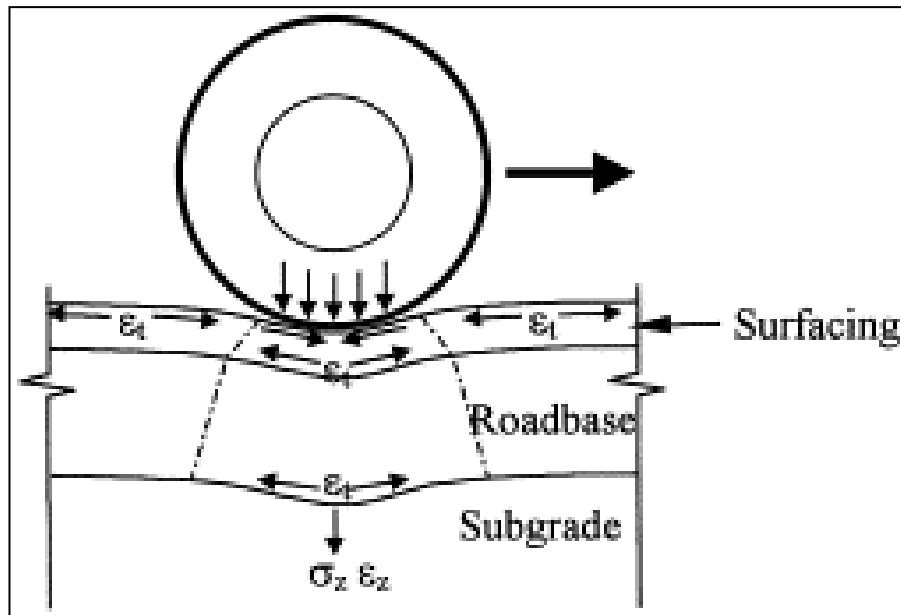


Figure 8.2 Critical Strains in Pavement Layers

The temperature distribution within the pavement structure, as shown in Figure 8.3 also has an effect on the magnitude of stresses.

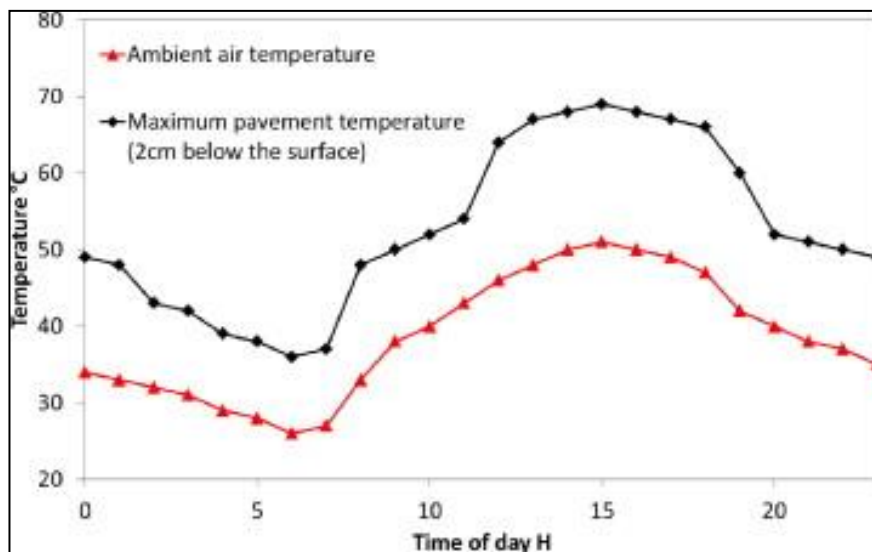


Figure 8.3 Example of Daily Temperature Variation in a Road Pavement

The strain criteria are those that generally limit the horizontal and vertical strains below those that will cause excessive cracking and excessive permanent deformation. These criteria are considered in terms of repeated load applications since it is known that the accumulated repetitions of the traffic loads are of

significant importance to the development of cracks and permanent deformation of the pavement.

8.1.4 Mechanistic Design Process

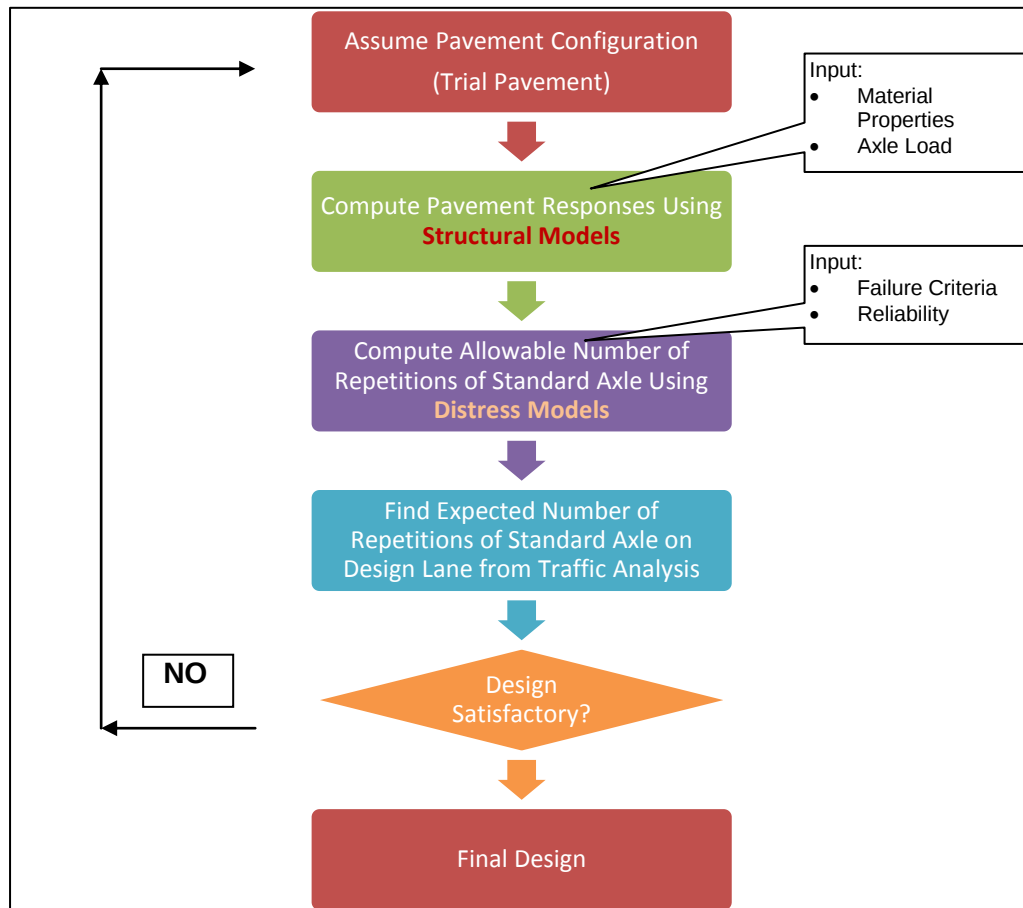


Figure 8.4 Flow Diagram of Mechanistic Design Procedure

1. Assume Pavement Configuration

This is done from one of the preceding methods. The design catalogue is often a good starting point to select a trial pavement.

2. Compute Pavement Response

A computer package is used to model the trial pavement to determine critical distress parameters. Table 8.4 shows the critical distress parameters that must be determined for each material type.

Table 8.4 Critical Distress Parameters

Material Type	Distress	Critical Distress Parameters
Thin hot-mix surfacing (20 - 75 mm)	Fatigue cracking	Horizontal tensile strain at bottom of the layer (ϵ_t)
Thick asphalt bases	Fatigue cracking	Horizontal tensile strain (ϵ_t)
	Deformation	Stress state in asphalt
Granular layers	Deformation	Stress state (σ_1, σ_2) in middle of layer
Cemented layers	Shrinkage cracking	Horizontal tensile strain (ϵ_t)
	Fatigue cracking (slab state)	Horizontal tensile strain (ϵ_t) in the layer
	Deformation (in the equivalent granular state)	Stress state (σ_1, σ_2) in middle of layer
Cemented layers	Crushing at the top of the level	Vertical stress at the top of the layer (σ_v)
Sub-grade	Deformation	Vertical compressive strain (ϵ_v) at top of layer

Several computer programs are available that can be used to compute the critical stresses and strains.

ELSYM5 is one such easily-available programme and is recommended for use.

3. Compute Allowable ESALs Using Distress Models

The Nigerian Empirical Mechanistic Pavement Analysis and Design System (NEMPADS) is a framework for mechanistic-empirical pavement design for tropical climate in Nigeria.

Murana and Olowosulu (2012) evaluated nine fatigue distress models, and seven rutting models for the Nigerian environment.⁽²⁵⁾

Table 8.5 Distress Models Evaluated

S/No	Models	Fatigue equation
1	AI model	$N_f = 0.0796(\varepsilon_t)^{-3.291}(E)^{-0.854}$
2	Shell model	$N_f = 0.0685(\varepsilon_t)^{-5.671}(E)^{-2.363}$
3	Belgian Road Research Center	$N_f = 4.92 \times 10^{-14}(\varepsilon_t)^{-4.76}$
4	UC-Berkeley Modified AI model	$N_f = 0.0636(\varepsilon_t)^{-3.291}(E)^{-0.854}$
5	Transport and Road Research Laboratory	$N_f = 1.66 \times 10^{-10}(\varepsilon_t)^{-4.32}$
6	Illinois model	$N_f = 5 \times 10^{-6}(\varepsilon_t)^{-3.0}$
7	U.S. Army model	$N_f = 478.63(\varepsilon_t)^{-5.0}(E)^{-2.66}$
8	Minnesota model	$N_f = 2.83 \times 10^{-6}(\varepsilon_t)^{-3.206}$
9	Indian model	$N_f = 0.1001(\varepsilon_t)^{-3.565}(E)^{-1.4747}$

Table 8.6 Rutting Models Evaluated

S/No	Models	Rutting equation
1	Iranian model	$N_r = 1.365 \times 10^{-9}(\varepsilon_v)^{-4.477}$
2	Indian model	$N_r = 2.56 \times 10^{-8}(\varepsilon_v)^{-4.5337}$
3	Minnesota model	$N_r = 5.5 \times 10^{15} \left(\frac{1}{\varepsilon_v \times 10^6} \right)^{-3.949}$
4	Federal Ministry of Works and Housing	$N_r = 1.66 \times 10^{-9} \left(\frac{1}{\varepsilon_v} \right)^{4.7037}$
5	Original shell model	$N_r = 2.3 \times 10^{-10} \left(\frac{1}{\varepsilon_v} \right)^{4.92}$
6	Asphalt institute	$N_r = \left(\frac{0.0105}{\varepsilon_v} \right)^{3.5714}$
7	Updated shell model	$N_r = \left(\frac{0.0105}{\varepsilon_v} \right)^{3.5714}$

Where

N_f = Number of allowable 8200 kg ESAL applications,

ϵ_t = Horizontal tensile strain at the bottom of the asphalt layer

E = dynamic modulus of the asphalt concrete in PSI.

N_r = Number of allowable 8,200 kg ESAL application.

ϵ_v = Vertical compressive strain at the top of the subgrade

The study concludes:

- Transport and Road Research Laboratory pavement performance model for fatigue's equation is a good predictor for 'NEMPADS' fatigue when considering high level of reliability and conservation.

$$N_f = 1.66 \times 10^{-10} (\epsilon_t)^{-4.32} \dots \text{Equation 8-3}$$

- The Indian equation should be used as a predictor for 'NEMPADS' pavement performance model for rutting for the facts that the environmental conditions of Nigeria is similar to that of India

$$N_r = 2.56 \times 10^{-8} (\epsilon_v)^{-4.5337} \dots \text{Equation 8-4}$$

4. Determine design traffic

Chapter 3 describes how design traffic is calculated. Overloading should be taken into consideration

5. Compare design traffic with capacity of structure as per '3'

Using the proposed distress models, the number of ESALs that the trial pavement can carry is compared with the actual design traffic

If the design traffic is greater than the capacity of the trial pavement, a new trial pavement is selected and the process repeated. The new trial pavement should have thicker layers or stronger material.

The process is repeated until a suitable pavement is found which has a capacity equal to or greater than the design traffic.

Note: in order to minimise costs, the capacity of the pavement selected should not exceed the design traffic by much – the pavement structure should be optimised.

8.2 Rigid Pavements

8.2.1 Portland Cement Association (PCA) Method

The designer is referred to the publication:

Thickness Design for Concrete Highway and Street Pavements, Portland Cement Association, 1984 (Reprinted 1995).

The design procedures given in this text apply to the following types of concrete pavements:

- **Plain Pavements:** constructed without reinforcing steel or doweled joints. Load transfer at the joints is achieved by aggregate interlock between the cracked faces below the joint saw cut or groove.
- **Plain Dowelled Pavements:** built without reinforcing steel, however, smooth steel dowel bars are used as load-transfer devices at each contraction joint and relatively short joint spacings are used to control cracking.
- **Reinforced Pavements:** contain reinforcing steel and dowel bars for load transfer at the contraction joints. The pavements are constructed with longer joint spacing than used for unreinforced pavements. Between the joints one or two transverse cracks will usually develop. These are held tightly together by the reinforcing steel and good load transfer is provided.
- **Continuously Reinforced Pavements:** built without contraction joints. Due to the relatively heavy steel reinforcement in the longitudinal direction, these pavements develop transverse cracks at close intervals. A high degree of load transfer is developed at these crack faces held tightly together by steel reinforcement.

9 Overlay Design

9.1 Introduction

Numerous overlay design methods are available all over the world. Here a brief introduction is made to three methods, indicating their main characteristics, assumptions and recommendations for their use in different environments.

9.2 The Asphalt Institute Method

9.2.1 Principles of the Method

The method was first published in 1969 by the Asphalt Institute.⁽²⁶⁾ The design manual covers both geometric and structural improvements of pavements in order to increase the traffic capacity, load carrying ability and the safety of the road user.

The manual identifies the following causes of a structurally inadequate pavement:

- Increase in traffic
- Change in the pavement material properties
- Inadequate design procedures.

Different causes of the pavement distress problem are identified, but no distinction is made between the recommended approaches to evaluation and rehabilitation design of any pavement. Two empirically derived procedures for the evaluation of structural adequacy and overlay design are given. These procedures are presented as applicable to all flexible pavements and any cause and mechanism of distress.

The recommended procedures are based on the pavement component analysis and pavement response analysis (surface deflection) approaches. Although seemingly different, both the recommended procedures are empirically derived and aim at providing adequate protection to the sub-grade of the pavement. In this case, the pavement rehabilitation problem is approached in a similar way to well-known empirical methods used for the design of new pavements, such as the CBR approach. Some subjective use of the existing condition of the pavement is made in the pavement component analysis procedure.

The main characteristics of the Asphalt Institute method are summarized in Table 9.1

Table 9.1 Main Characteristics of Asphalt Institute Method

DEVELOPER	BASIS OF DERIVATION	CLASSIFICATION ACCORDING TO APPROACH	PAVEMENT EVALUATION SURVEYS	APPLICABILITY	INPUT TO DESIGN	LIMITATIONS (main)	ADVANTAGES (main)
Asphalt Institute in the USA	Empirical	Pavement response analysis 1 Deflection-based 2 Pavement component analysis	1. Surface deflections 2. CBR of sub-grade Type, thickness and condition of pavement layers	- Developed with data from pavements with granular sub-layers and thin surfacing - Pavement component analysis method incorporates all types of material using conversion factors and subjective condition ratings	- Surface deflection - Temperature - Past traffic loading - Future traffic loading - CBR of sub-grade (DSS) - Type, condition, thickness of pavement layers - Past traffic loading - Future traffic loading	- Empirically base with limitations to applicability - Design based on protection of sub-grade only - No seasonal adjustment factors given - AI curve applicable to traffic loading up to $7,3 \times 10^8$ E80s - Component analysis procedure applicable up to $36,5 \times 10^6$ E80s	Based on easy-to-apply and well-known concepts
ASSUMPTIONS		MATERIAL CHARACTERIZATION	RESIDUAL LIFE	BASIS OF DESIGN LIFE	PAVEMENT TRANSFER FUNCTIONS CONSIDERATION		
- Those applicable to deflection-based methods - Deflection-overlay thickness relation incorporates assumptions of linear elasticity theory - Those applicable to pavement component analysis method - The cover requirement of the pavement is independent of the requirements of the individual pavement layers. - All pavement materials can be converted to an equivalent asphalt thickness.	1. - 2. Uses subjective assessment of the condition of pavement layers	1. Use of deflection versus life curve 2. Uses nomograph relating pavement equivalent thickness to life	- Protection of the sub-grade - Pavement deformation	1. Deflection of structure			

9.3 Transport and Road Research Laboratory (TRRL) Surface Deflection Method

9.3.1 Principles of the Method

The pavement evaluation and rehabilitation design method developed at the Transport and Road Research Laboratory (TRRL) was published in 1978.⁽²⁷⁾ The method is empirically derived and uses surface deflection as a design parameter. It was based on many years of research and experimental work on roads dating back to 1956. The main objective of the method is to provide a system for the design of pavement-strengthening measures that will enable the engineer to:

- Predict the remaining life of a pavement before a critical condition is reached
- Design the thickness of overlay required to extend the life of the pavement to carry a given design traffic.

The method is based on the characterisation of the structural condition of the pavement through the measurement of surface deflection. Although all measurements were based on deflections under a dual wheel single axle load of 6 350 kg moving at creep speed, this can easily be adapted to deflections measured under a 80 kN (8 175 kg) dual wheel single axle load. The design manual pays special attention to the analytical procedures and the correct measurements, adjustment and use of pavement surface deflections in the various design charts. The method also takes into account and distinguishes between deflections measured on the main types of road base. The recommended method entails the following:

- i. measurement of the surface deflection of the pavement
- ii. adjustment of the measured deflection
- iii. estimation of past, present and future expected traffic on the pavement
- iv. prediction of the residual bearing capacity of the pavement before reaching a critical condition
- v. design of an overlay to increase the bearing capacity of the pavement to carry the design traffic
- vi. matching the overlay design to variations in the structural condition of the pavement.

The main characteristics of the TRRL deflection method are summarised in Table 9.2

Table 9.2 Main Characteristics of the TRRL Surface Deflection Method

DEVELOPED	BASIS OF DERIVATION	CLASSIFICATION ACCORDING TO APPROACH	PAVEMENT EVALUATION SURVEYS	APPLICABILITY	INPUT TO DESIGN	LIMITATIONS (main)	ADVANTAGES (main)
United Kingdom (TRRL)	Empirical	Pavement response analysis (deflection based)	- Surface deflections - Temperature	- Design charts developed for main types of flexible pavement - Some limitations of use for pavements with cemented layers	- Surface deflections - Temperature at time of measurements - Thickness of asphalt layer - Type of pavement - Past traffic loading - Future traffic loading	- Empirically based with limitations in applicability - No seasonal adjustment given - Applicable for traffic loading up to 10×10^6 E80s - Design (except for pavement with cemented layers) based on deformation originating from the sub-grade only	- Base on easy NDT testing - Is easy to use - Distinguishes between the types of pavement - Identifies some limitations in the use of the method on pavements with cemented layers - Adjust deflections to take the effect of temperature variations into account
ASSUMPTIONS		MATERIALS CHARACTERISATION	RESIDUAL LIFE	BASIS OF DESIGN LIFE	PAVEMENT TRANSFER FUNCTIONS CONSIDERED		
- Those applicable to deflection-based methods - Pavement life is a function of type of pavement and surface deflection			Use of deflection versus life curves	Deformation in the form of rut depth except for pavements with cemented layers where cracking is used as a criterion	Deflection of structure		

9.4 The Shell Overlay Design Method

9.4.1 Principles of the Method

The Shell overlay design method was developed at the Koninklijke Shell Laboratory in Amsterdam, from the Shell design procedure for new pavements. The charts originally developed for the design of new pavements are also used for the design of asphalt overlays for existing pavements. Non-destructive test (NDT) methods are used to assess the properties of the existing pavement.⁽²⁸⁾

In the Shell method, a number of design charts are used to determine the required thickness of overlays for the rehabilitation of a pavement. The design charts were derived from the results of analyses of pavements by means of the theory of linear elasticity. In these analyses, the pavement was assumed to be adequately represented by a three-layered model consisting of a top layer (surfacing) of asphaltic material, a middle layer (base) of granular or cementitious material and a bottom layer (sub-grade) of semi-infinite dimensions as shown in Figure D4₉₂. In this model the pavement materials are characterised by the following properties:

- Surfacing layer - the effective modulus of deformation (E1 value), a Poisson's ratio (ν_1) and a layer thickness (h_1),
- Base layer - the effective modulus of deformation (E2 value), a Poisson's ratio (ν_2) and a layer thickness (h_2), and
- Sub-grade layer - the effective modulus of deformation (E3 value) and a Poisson's ratio (ν_3) (the layer thickness is assumed to be infinite).

In the development of the method, the BISAR computer program was used to calculate the stresses and strains in pavement structures. The results obtained were used to compile the design charts. The primary design criteria used for the compilation of these charts were:

- The compressive vertical strain in the surface of the sub-grade which controls deformation of the sub-grade material
- The horizontal tensile strain in the asphalt layer which controls cracking of the asphalt layer
- The tensile stress of strain in any cementitious base layer which controls cracking of the cementitious layer

In the evaluation of pavements using the derived charts, fixed values are assumed for the Poisson's ratios for all the layers. Falling Weight Deflectometer (FWD) measurements, material tests and available data are used to determine the other in-situ properties. The maximum deflection and the shape of the deflection bowl, as measured by the FWD, are used to determine the sub-grade modulus (E_3) and the effective thickness of the asphalt layer (h_1). The deflection bowl is characterised by the ratio (Q_r) of the deflection at a distance ' r ' from the load (s_r) to the deflection under the centre of the test load (s_o).

The distance r should preferably be such that $Q_r \sim 0.5$. The design charts ensure that the strains (mentioned above) are limited to such an extent that virtually no cracking will occur in the structure, and that there will be no excessive permanent deformation in the sub-grade during the design life of the pavement.

Practical constraints made it impossible to develop design charts for every conceivable pavement configuration. Hence the method will often require design charts to be interpolated for overlay design. The use of such charts eliminates the need to calculate the critical parameters of the pavement through computer simulation of the pavement response.

Although the design approach is based on the abovementioned strain criteria, the Shell method also provides for the testing of the expected permanent deformation in the asphalt layer, and for checking of the maximum overlay thickness. Provision is made to include climatic variations (temperature changes) in the design of the overlay.

Procedures included in the method take into account the variations in asphalt mix properties available for overlays, and assess their differences in comparison with the asphalt on the existing pavement, and their influence on the expected future behaviour of the rehabilitated pavement.

The main characteristics of the Shell overlay design method are summarised in Table 9.3.

Table 9.3 Main Characteristics of Shell Overlay Design Method

DEVELOPED	BASIS OF DERIVATION	CLASSIFICATION ACCORDING TO APPROACH	PAVEMENT EVALUATION SURVEYS	APPLICABILITY	INPUT TO DESIGN	LIMITATIONS (main)	ADVANTAGES (main)
The Netherlands (Koninklijke Shell Laboratorium Amsterdam)	Theoretical (linear elasticity theory)	Application through the use of design charts	Falling weight Deflectometer (FWD)	All types of flexible pavements (special provision for cementitious layers)	- Maximum deflection - Deflection ratio (Or) - Temperature	- Incorporates only temperature changes - as a climate effect - Does not assess unbound pavement layers - Charts based on asphalt fatigue and sub-grade deformation only	- Bases on NDT tests - Tests overlay mix properties into account - Incorporates and allows for temperature gradients in the asphalt layer - Allows for the checking of deformation in asphalt layers - Incorporates different climates through w-MAAT factor
ASSUMPTIONS			MATERIALS CHARACTERISATION	RESIDUAL LIFE	BASIS OF DESIGN LIFE	PAVEMENT TRANSFER FUNCTIONS CONSIDERED	
- Those applicable to linear elasticity theory - Distress = f (asphalt strain, sub-grade strain) - Modulus of base is a function of modulus of sub-grade and the thickness of the base layer - Poisson's ratio for all layers = 0,35			Use NDT (FWD) measurements - maximum deflections (o) - deflection ratio (Or) - temperature - thickness of base (h₂)	Use FWD measurement between the wheel tracks to determine as built pavement life from design charts	- Tensile strain in asphalt - Compressive strain in sub-grade - Viscous-nature of asphaltic material	- Deflection basin - Asphalt tensile strain - Sub-grade compressive stress	

10 Practical Considerations

10.1 Materials Availability

Bearing in mind that the cost of transport of materials becomes a major cost factor if materials must be brought in to the site from a distance, it is usually cost-effective to try to utilise the local materials even if this would then necessitate some form of processing. This may take various forms, but the choice is, of course, ostensibly a matter of cost and economy and in most cases the pavement designer must select materials accordingly.

10.2 Terrain

The performance of a road in otherwise similar conditions can be influenced by terrain, in that rolling or mountainous terrain (in which significant grades are encountered) tends to lead to significantly more traffic-related loading on surfacings and bases. This is fairly commonly observed on relatively heavily trafficked roads (say, class T5 and higher, carrying more than 3 million ESAs), where surface deterioration and rutting deformation occurs. Routes on which overloaded trucks are common (axle loads of 10 tonnes and more) are especially prone.

In such situations, it is imperative that compaction of layers is controlled extremely well and ideally to more than minimum standards. It is also advised that the surfacing layer is resistant to deformation and of course, well-bonded to the base to avoid early failure due to debonding and traffic-induced slippage at the interface.

A bituminous base combined with a hot mix asphalt surfacing can be (and is often) used to provide a stable, relatively stiff, deformation resistant backbone, which can also mask possible compaction deficiencies in the underlying layers, which may occur due to difficult working conditions. There is considerable merit in looking at the use of special bituminous binders which may help inhibit rutting due to heavy vehicles, and the guidance of the bitumen supplier should be sought in the first instance.

An alternative approach, not specifically covered in this guide, is to consider the possibility of a concrete base. This type of construction can be effective for these

conditions, and can be laid by labour-enhanced methods where conventional large-scale construction equipment is unsuitable.

It is also commonly observed that moisture-induced problems, leading to possible local premature failures, occur in cuts and on sag curves (dips), emphasising the need for particular attention to drainage provision and maintenance in such locations.

10.3 Vehicle Overloading

Incidences of vehicle overloading can have a significant negative impact on the performance of a road, and the effects are observed especially by premature failures of surfacing layers (excessive rutting, bleeding, loss of surface texture, and ravelling being prevalent as early indicators). Naturally, every effort should be made to limit the amount of overloading (illegal loading), but it is recognised that current controls may not always be sufficient.

While the design process should account for the amount of heavy vehicle axle loads in determining the design traffic loading, the specific effects of the very heavy abnormal axle loads on the pavement must be considered in finalising the design.

In situations where overloading is likely to occur, special attention must be given to the quality and strength of all the pavement layers during construction. Amongst other measures, there may be justification in increasing the specification CBR requirements for granular layers, in increasing the base and sub-base layer thicknesses, and in specifying special bitumen binders and asphalt mixtures, such as stone mastic asphalts, which are more resistant to deformation.

The specific measures that the Engineer may deem necessary should ideally be based on either proven local practice or at least specialised advice/analysis in order to maintain a well-balanced structure.

10.4 Subgrade California Bearing Ratio (CBR) less than two per cent

In these cases, which must be treated according to the specific situation, some of the possible approaches include:

- In- situ treatment with lime (for clayey materials)
- Removal and replacement with better quality material
- Use of geofabrics
- Construction of a pioneer layer (for highly expansive material and marshy areas) or rockfill

These conditions are often encountered in low-lying, wet and swampy areas, and treatment should ideally be based on past proven practice for similar conditions. The use of geofabrics, usually in accordance with specialist advice from the manufacturer, can be extremely effective in situations where other approaches are inappropriate (for example, where better quality materials are either not readily available, or would tend to displace downwards).

When appropriately treated, the design for the overlying pavement can then be based on the re-evaluated subgrade support condition.

10.5 Use of the Dynamic Cone Penetrometer (DCP)

The DCP is probably the single most effective testing device for road construction, being a simple, rapid and direct indicator of material condition that can be used from initial site survey through to construction control. Its use within the region is already established, and this section is intended only to highlight the main aspects of its effective usage.

During initial field survey, the DCP can aid in determining the existing subgrade condition, in conjunction with normal indicator and CBR tests, and therefore in delineating uniform sections for design. Similarly, during construction the DCP can be used to monitor uniformity of layers, particularly in terms of in- situ density. It can also be used as a design tool in its own right and a method has been developed for such application⁵.

While the DCP is commonly used to estimate in-situ CBRs from nominal penetration rates (mm/blow), this technique should only be used when correlations have been specifically developed for the DCP apparatus used. It is known that several different types of DCP are commonly used, having different cone types and dynamic energy input. If used with the wrong CBR correlations, incorrect estimates of CBR will be obtained. Since changes in moisture content will influence the rate of penetration for a given density, the Engineer must ensure that this factor is taken into account if the DCP is used for CBR estimation.

DCP (Dynamic Cone Penetrometer) testing shall be carried out at intervals of 200 meters to directly measure the field CBR strength of the subgrade. Continuous measurements can be made down to a depth of approximately 1000mm or when extension shafts are used to a recommended maximum depth of 2 meters. The interval of test pits for CBR shall not be more than 1.0 km. Correlations have been established between measurement with the DCP and CBR, so that results can be interpreted and compared with CBR specifications for pavement design.

Alternatively, and especially for control monitoring, the penetration rate can be used in its own right as a compliance check. For example, the Engineer can determine an acceptable maximum DCP penetration rate directly from in-situ measurements on areas (of subgrade or constructed granular layers) deemed to meet the required field strength and density requirement. The DCP can then be used as a process control tool to check that the field compaction is satisfactory to the specified depth. Where penetration rates exceed the acceptable specified maximum value, further compaction is indicated.

The DCP should not be used specifically, however, as the basis for determining construction acceptance (i.e. for density or strength compliance with the specification requirements); this should still be undertaken using the appropriate standard test methods.

Consequently, the use of the DCP during the whole construction process from initial field survey, through to rapid compliance checking, can significantly reduce the need for some of the more onerous testing, and its use is strongly recommended.

10.6 Performance Records

The experience and judgment of the highway engineer is based to a large degree on the performance of pavements in the immediate area of his jurisdiction. Past performance is a valuable guide of conditions and service requirements for the reference pavements, when the conditions are comparable to those for the designs under study. Caution is urged however, against reliance on short-term performance records, and on long term records of pavements which may have been subjected to much lighter loadings for a large portion of their present life.

Climate may have a significant effect on pavement performance, and must be carefully considered in evaluation performance records from other regions.

10.7 Secondary Factors

The following factors may also influence pavement selection:

- Traffic safety
- Availability of local materials
- Adjacent existing pavements
- Stage construction
- Conservation of aggregate
- Other construction considerations

10.8 Skid Resistance

Skid resistance is the force developed when a tire that is prevented from rotating slides along the pavement surface (Highway Research Board, 1972). Skid resistance is an important pavement evaluation parameter because:

- Inadequate skid resistance will lead to higher incidences of skid related accidents.
- Most agencies have an obligation to provide users with a roadway that is "reasonably" safe.
- Skid resistance measurements can be used to evaluate various types of materials and construction practices.

Skid resistance depends on a pavement surface's microtexture and macrotexture (Corley-Lay, 1998). Microtexture refers to the small-scale texture of the pavement aggregate component (which controls contact between the tire rubber and the pavement surface) while macrotexture refers to the large-scale texture of the pavement as a whole due to the aggregate particle arrangement (which controls the escape of water from under the tire and hence the loss of skid resistance with increased speed) (AASHTO, 1976). Skid resistance changes over time. Typically it increases in the first two years following construction as the roadway is worn away by traffic and rough aggregate surfaces become exposed, then decreases over the remaining pavement life as aggregates become more polished.

Skid resistance is generally quantified using some form of friction measurement such as a friction factor or skid number.

Friction factor (like a coefficient of friction): $f = F/L$

Skid number: $SN = 100(f)$

where: F = frictional resistance to motion in plane of interface

L = load perpendicular to interface

It is not correct to say a pavement has a certain friction factor because friction involves two bodies, the tires and the pavement, which are extremely variable due to pavement wetness, vehicle speed, temperature, tire wear, tire type, etc. Typical friction tests specify standard tires and environmental conditions to overcome this.

In general, the friction resistance of most dry pavements is relatively high; wet pavements are the problem. The number of accidents on wet pavements are twice as high as dry pavements (but other factors such as visibility are involved in addition to skid resistance). Table 10.1 shows some typical Skid Numbers (the higher the SN, the better).

Table 10.1: Typical Skid Numbers⁽²⁹⁾

Skid Number	Comments
< 30	Take measures to correct
≥ 30	Acceptable for low volume roads
31 - 34	Monitor pavement frequently
≥ 35	Acceptable for heavily travelled roads

Skid testing may occur in a number of ways:

- The locked wheel tester
- The spin up tester
- Surface texture measurement

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Appendices

APPENDIX A: Nigerian Traffic and Axle Load Study

Nigerian Traffic AND axle Load Study

A 1.1 Background

The data used in this section is obtained from an extensive survey that was carried out as part of an axle-load study completed in 2008.

The surveys provide valuable information about the traffic loading on the Nigerian Federal Road Network.

A key finding of the study was that overloading is rife in Nigeria, which has serious implications on the performance and durability of pavements. This will be incorporated into this guideline document.

A 1.2 Representative Traffic Flows on Federal Road Network

Table A.1 shows the percentage of heavy vehicles on the Federal Road network. This is useful for the estimation of traffic on roads where detailed information is unavailable.

Table A.1: Percentage Heavy Vehicles on Road Network

Major Federal Road Link	ADT	Heavy Vehicles /Day	% Heavy Vehicles
Lagos - Shagamu	40 000	5 000	13%
Shagamu - Benin City	22 000	3 100	14%
Shagamu - Ibadan	8 900	2 800	31%
Benin - Warri - Port Harcourt	5 000	350	7%
Port Harcourt - Aba	18 000	2 200	12%
Aba - Enugu	12 000	2 000	17%
Aba - Nlugu	8 900	2 000	22%
Nlugu - Calabar	4 500	1 000	22%
Enugu - Nkalagu	6 000	1 200	20%
Nkalagu - Mfom	4 000	500	13%
Benin City - Onitsha	14 500	2 500	17%
Onitsha - Enugu	18 000	1 500	8%
Benin City - Lokoja	7 300	1 000	14%
Ibadan - Ilorin	10 000	2 500	25%
Ilorin - Jebba	5 000	2 200	44%
Mokwa-Bida	4 500	1 500	33%

Major Federal Road Link	ADT	Heavy Vehicles /Day	% Heavy Vehicles
Bida - Abuja	2 200	550	25%
Lokoja - Abuja	9 000	900	10%
Abuja - Kaduna	8 000	800	10%
Abuja - Akwanga	5 700	250	4%
Enugu - Makurdi	6 000	920	15%
Makurdi - Akwanga	6 200	400	6%
Akwanga - Jos	4 000	220	6%
Jos - Bauchi	7 000	380	5%
Bauchi - Yola	4 200	370	9%
Kaduna - Zaria	11 000	920	8%
Zaria - Sokoto	5 100	420	8%
Sokoto - Illela	3 000	100	3%
Zaria - Kano	10 000	700	7%
Kano - Katsina	5 600	630	11%
Kano - Potisku	4 000	300	8%
Potisku - Maiduguri	5 000	920	18%
Maiduguri - Ngala	3 000	1 000	33%

As per the data in Table 3.5, the proportion of heavy vehicles on the network range between 3% - 33%, showing a big variation on Nigeria's Federal Road network.

Figure 11.1 schematically shows the volume of traffic on Federal Roads as Average Daily Traffic (ADT/ VPD)

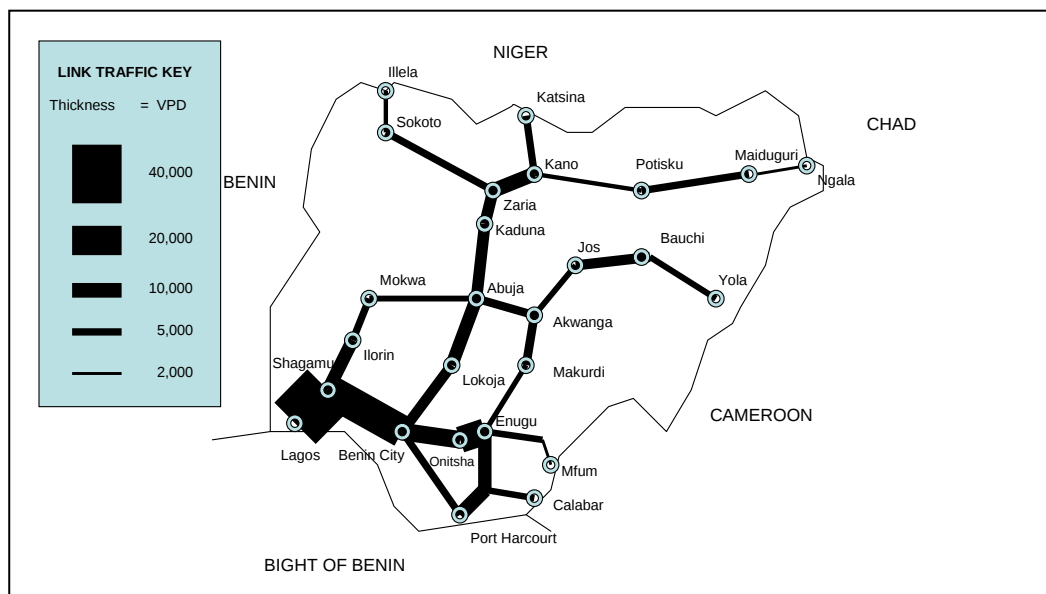


Figure 11.1 Link Traffic Flows (ADT) on Federal Road Network

A 1.3 2008 Axle-Load Study: Independent Axle-Load Survey Positions

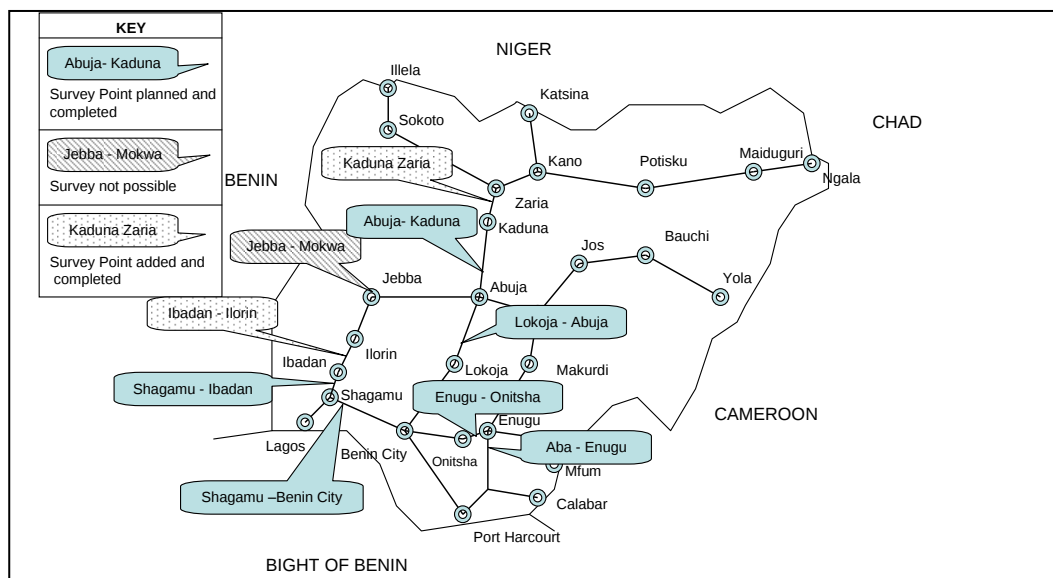


Figure 11.2 Axle-Load Survey Positions (2008)

A 1.4 ESAs per Heavy Vehicle

From the axle information collected in the 2008 study, average ESAs can be calculated for use in design traffic estimates.

Table A.2 shows the average values for laden and unladen vehicles.

Table A.2: ESAs per Heavy Vehicle based on Loading

Loading of Heavy Vehicles	ESAs/ Heavy Vehicle (Without Overloading)	ESAs/ Heavy Vehicle (With Overloading)
Mostly Unladen	0.6	-
Both Laden and Unladen	1.6	10.0
Mostly Laden	2.0	14.0

Table A.3 shows the average values by truck type.

Table A.3: ESAs per Heavy Vehicle based on Truck Type

Truck Type	Average ESALs per vehicle (Without Overloading)	Average ESALs per vehicle (With Overloading)	Range in ESALs per vehicle found at different sites
2-Axle	1.0	5.5	2.0 - 25.0
3-Axle	1.5	14.6	6.1 - 48.0
4-Axle	2.5	22.8	17.4 - 59.7
5-Axle	1.8	19.6	14.0 - 43.2
6-Axle	1.2	8.5	5.2 - 21.5

APPENDIX B: Nigerian subgrades

B 1: Nigerian Subgrades

B 1.1 General

The information contained in this section was obtained from research reports obtained from the Nigerian Building and Road Research Institute. The information is not intended to replace normal engineering process, including materials testing at a specific site. Rather, the information is intended to provide the designer with additional information that may be used for comparison and preliminary preparation for design.

Unfortunately, the available information is not exhaustive and only covers part of Nigeria. This section would thus be updated as and when information became available.

B 1.2 Engineering Properties of Subgrade Soils in the Federal Capital Territory

The region is underlain by the basement complex consisting of crystalline rocks. The major rock types encountered in the area include:

- a) Igneous Rocks- fine to coarse-grained granites are the predominant rocks in the area
- b) Metamorphic Rocks- these are mainly migmatites and migmatite gneiss, and schists rich in flaky minerals that are easily susceptible to weathering due to high foliation
- c) Sedimentary Rocks- consisting mostly of sand with gravel beds and clay deposits

Classification of Subgrades in Federal Capital Territory

The tables below show the percentage distribution of subgrade soils in the Federal Capital Territory:

Unified Classification System	
Subgrade Soil Group	Percentage Distribution (%)
SM	8
SC	48
ML	2
CL	14
MH	8
CH	20

AASHTO Classification	
Subgrade Soil Group	Percentage Distribution (%)
A-2-4	2
A-2-6	8
A-2-7	16
A-6	14
A-7-5	10
A-7-6	50

B 1.3 Engineering Properties of Subgrade Soils in Imo State

The general topography of Imo State is characterised by gullied hill slopes underlain by unconsolidated sedimentary rocks which date back to the Upper Cretaceous. The general lithology in which Imo State lies consists of the following:

- Alternation of coarse sands with clays and shales
- Clays with lignite
- Grey clayey sandstone and sandy claystone
- Laminated clayey shales
- Shales with coal and sandstone beds

Classification of Subgrades in Imo State

The tables below show the percentage distribution of subgrade soils in Imo State:

Unified Classification System	
Subgrade Soil Group	Percentage Distribution (%)
SC	70
SM	3
SM-SC	5
CL	8
CH	8
MH or OH	5

AASHTO Classification	
Subgrade Soil Group	Percentage Distribution (%)
A-2-4	15
A-2-6	28
A-2-7	8
A-6	7
A-7-5	8
A-7-6	33

B 1.4 Engineering Properties of Subgrade Soils in Bendel (Delta and Edo) State

About 90% of the state is underlain by sedimentary rocks, while the remaining 10% located in the northern-most part around Igarra, is underlain by crystalline rocks of the Basement Complex.

The existing crystalline rocks are mainly metamorphic and consist of:

- Migmatite-gneiss complex
- Undifferentiated metasediments made up of schists and quartzite
- Porphyritic older granites
- Other non-metamorphosed syenite dykes

These rocks can be quarried for use as aggregates in road construction.

Classification of Subgrades in Bendel State

The tables below show the percentage distribution of subgrade soils in Bendel State:

Unified Classification System	
Subgrade Soil Group	Percentage Distribution (%)
GC	1
SM	7
SC	53
SM-SC	9
CH	9
CL	15
CL-ML	1
MH-OH	1
ML-OL	3

AASHTO Classification	
Subgrade Soil Group	Percentage Distribution (%)
A-2-4	16
A-2-6	18
A-2-7	4
A-4	9
A-6	13
A-7-5	4
A-7-6	36

APPENDIX C: Pavement Design Catalogue

The Catalogue contained herein is taken from Overseas Road Note 31.⁽⁶⁾

KEY TO STRUCTURAL CATALOGUE

Traffic classes

(10⁶ esa)

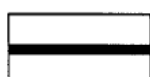
T1 =	< 0.3
T2 =	0.3 - 0.7
T3 =	0.7 - 1.5
T4 =	1.5 - 3.0
T5 =	3.0 - 6.0
T6 =	6.0 - 10
T7 =	10 - 17
T8 =	17 - 30

Subgrade strength classes

(CBR%)

S1 =	2
S2 =	3 , 4
S3 =	5 - 7
S4 =	8 - 14
S5 =	15 - 29
S6 =	30+

Material Definitions



Double surface dressing



Flexible bituminous surface



Bituminous surface
(Usually a wearing course, WC, and a basecourse, BC)



Bituminous roadbase, RB



Granular roadbase, GB1 - GB3



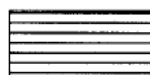
Granular sub-base, GS



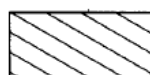
Granular capping layer or selected subgrade fill, GC



Cement or lime-stabilised roadbase 1, CB1

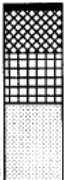
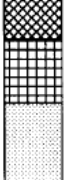
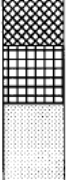
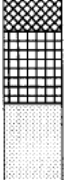
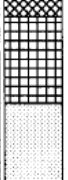
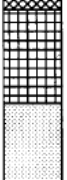
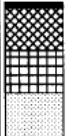
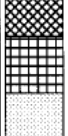
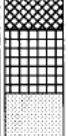
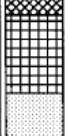
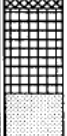
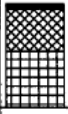
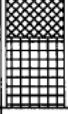
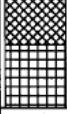
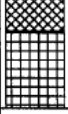
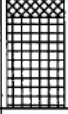
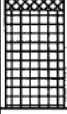


Cement or lime-stabilised roadbase 2, CB2



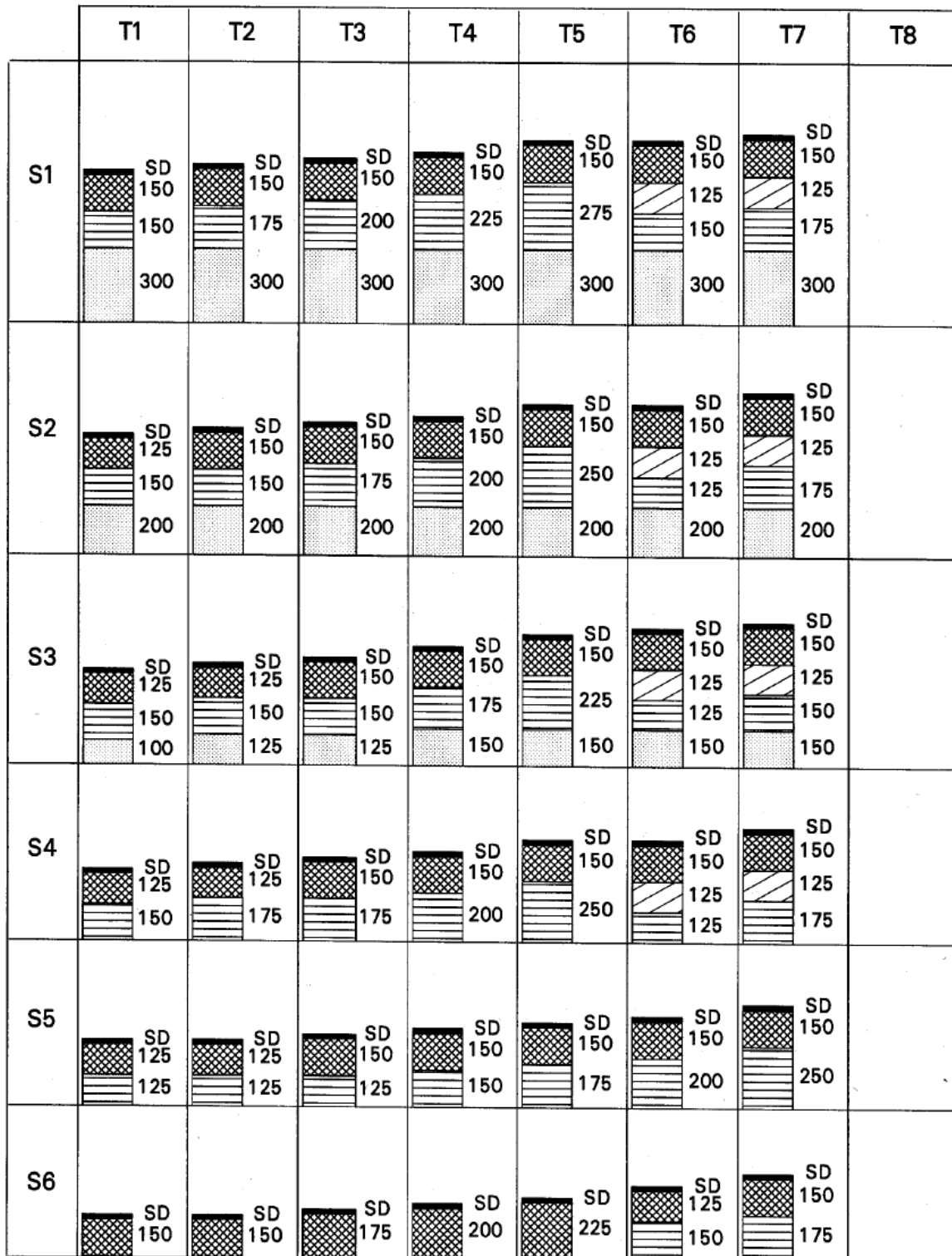
Cement or lime-stabilised sub-base, CS

CHART 1 GRANULAR ROADBASE / SURFACE DRESSING

	T1	T2	T3	T4	T5	T6	T7	T8
S1	 SD 150 175 300	 SD 150 225* 300	 SD 200 200 300	 SD 200 250* 300	 SD 200 300* 300	 SD 225 325* 300		
S2	 SD 150 150 200	 SD 150 200 200	 SD 200 175 200	 SD 200 225* 200	 SD 200 275* 200	 SD 225 300* 200		
S3	 SD 150 200	 SD 150 250	 SD 200 225	 SD 200 275* 200	 SD 200 325* 200	 SD 225 350* 200		
S4	 SD 150 125	 SD 150 175	 SD 200 150	 SD 200 200	 SD 200 250	 SD 225 275		
S5	 SD 150 100	 SD 150 100	 SD 175 100	 SD 200 125	 SD 225 150	 SD 250 175		
S6	 SD 150	 SD 150	 SD 175	 SD 200	 SD 225	 SD 250		

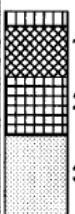
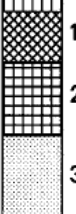
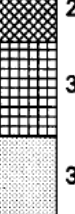
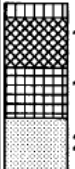
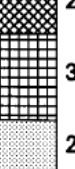
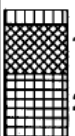
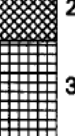
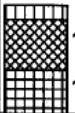
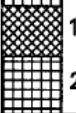
- Note: 1 * Up to 100mm of sub-base may be substituted with selected fill provided the sub-base is not reduced to less than the roadbase thickness or 200mm whichever is the greater. The substitution ratio of sub-base to selected fill is 25mm : 32mm.
- 2 A cement or lime-stabilised sub-base may also be used.

CHART 2 COMPOSITE ROAD BASE (UNBOUND & CEMENTED) / SURFACE DRESSING



Note: Sub-base to fill substitution not permitted.

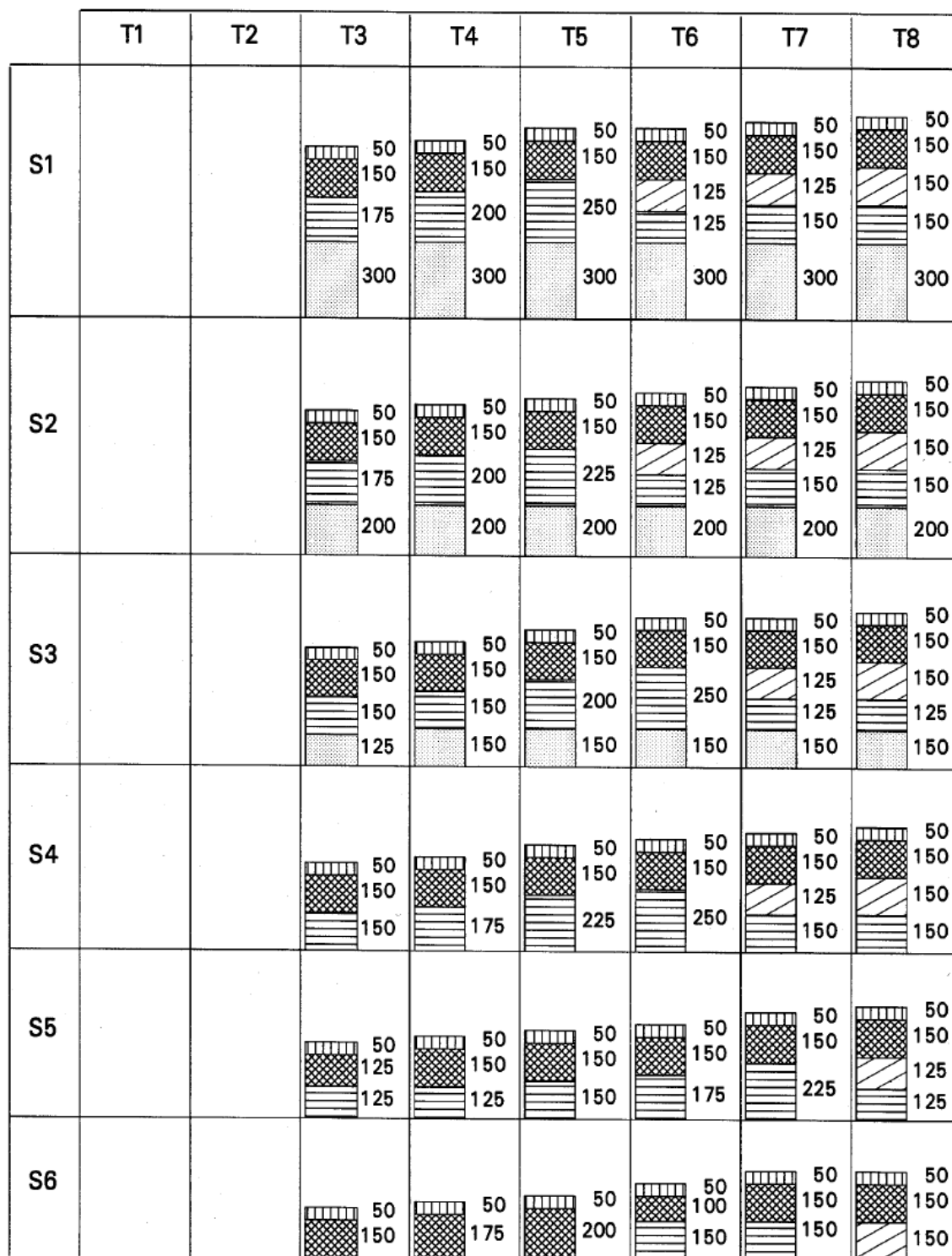
CHART 3 GRANULAR ROADBASE / SEMI-STRUCTURAL SURFACE

	T1	T2	T3	T4	T5	T6	T7	T8
S1			 50 175 200 300	 50 175 250* 300	 50 175 300* 300	 50 200 325* 300		
S2			 50 175 175 200	 50 175 225* 200	 50 175 275* 200	 50 200 300* 200		
S3			 50 175 225	 50 175 275* 200	 50 175 325* 200	 50 200 350* 200		
S4			 50 175 150	 50 175 200	 50 175 250	 50 200 275*		
S5			 50 150 100	 50 175 125	 50 175 150	 50 200 175		
S6			 50 150	 50 175	 50 200	 50 225		

Note: 1 * Up to 100mm of sub-base may be substituted with selected fill provided the sub-base is not reduced to less than the roadbase thickness or 200mm whichever is the greater. The substitution ratio of sub-base to selected fill is 25mm : 32mm.

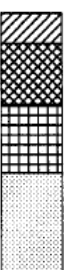
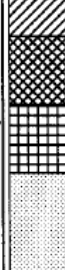
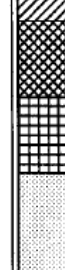
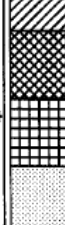
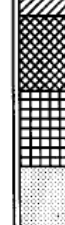
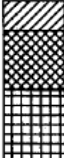
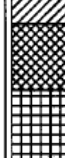
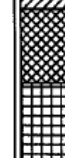
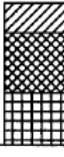
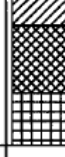
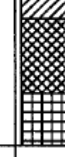
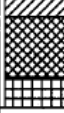
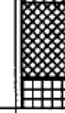
2 A cement or lime-stabilised sub-base may also be used.

CHART 4 COMPOSITE ROADBASE / SEMI - STRUCTURAL SURFACE



Note: Sub-base to fill substitution not permitted.

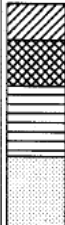
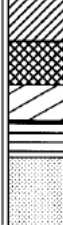
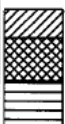
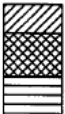
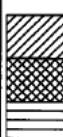
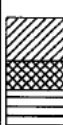
CHART 5 GRANULAR ROADBASE / STRUCTURAL SURFACE

	T1	T2	T3	T4	T5	T6	T7	T8
S1						 100 200 225* 350	 125 225 225 350	 150 250 250 350
S2						 100 200 225* 200	 125 225 225 200	 150 250 250 200
S3						 100 200 250	 125 225 250	 150 250 275
S4						 100 200 175	 125 225 175	 150 250 175
S5						 100 200 100	 125 225 100	 150 250 100
S6						 100 200	 125 225	 150 250

Note: 1 * Up to 100mm of sub-base may be substituted with selected fill provided the sub-base is not reduced to less than the roadbase thickness or 200mm whichever is the greater.
The substitution ratio of sub-base to selected fill is 25mm : 32mm.

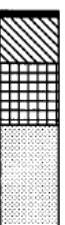
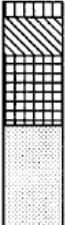
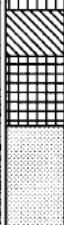
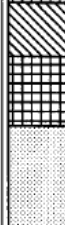
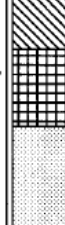
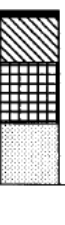
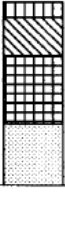
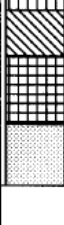
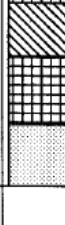
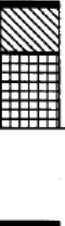
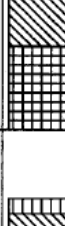
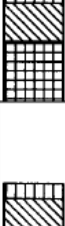
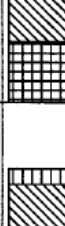
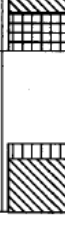
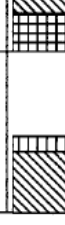
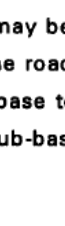
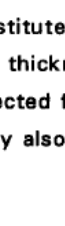
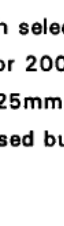
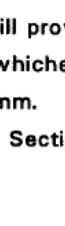
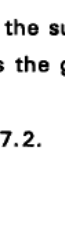
2 A cement or lime-stabilised sub-base may also be used.

CHART 6 COMPOSITE ROADBASE / STRUCTURAL SURFACE

	T1	T2	T3	T4	T5	T6	T7	T8
S1						 100 150 200 350	 125 150 250 350	 150 150 125 125 350
S2						 100 150 200 200	 125 150 250 200	 150 150 125 125 200
S3						 100 150 175 125	 125 150 200 125	 150 150 225 125
S4						 100 150 175	 125 150 200	 150 150 225
S5						 100 150 150	 125 150 150	 150 150 150
S6						 100 100 150	 125 100 150	 150 100 150

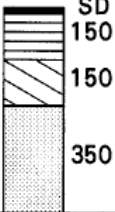
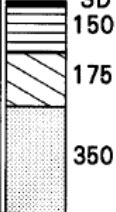
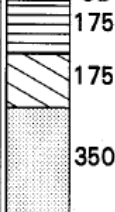
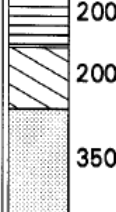
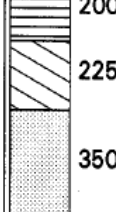
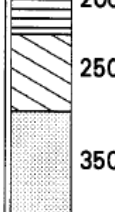
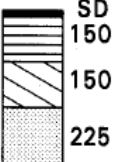
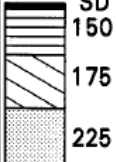
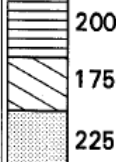
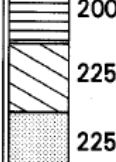
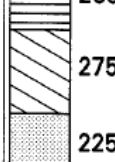
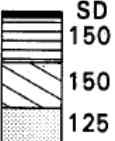
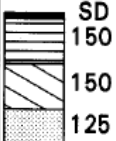
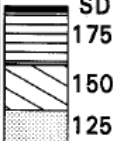
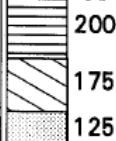
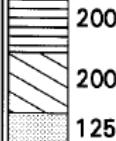
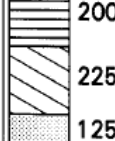
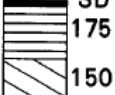
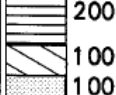
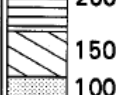
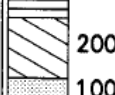
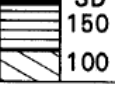
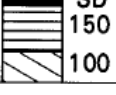
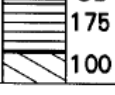
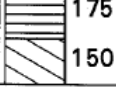
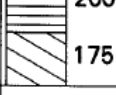
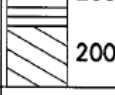
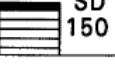
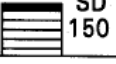
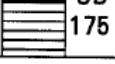
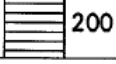
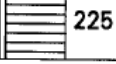
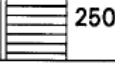
Note: Sub-base to fill substitution not permitted.

CHART 7 BITUMINOUS ROADBASE / SEMI-STRUCTURAL SURFACE

	T1	T2	T3	T4	T5	T6	T7	T8
S1				 SD 150 200 350	 SD 50 125 225* 350	 SD 50 150 225* 350	 SD 50 175 225* 350	 SD 50 200 250* 350
S2				 SD 150 200 200	 SD 50 125 225* 200	 SD 50 150 225* 200	 SD 50 175 225* 200	 SD 50 200 250* 200
S3				 SD 150 250 250	 SD 50 125 250	 SD 50 150 275* 275*	 SD 50 175 275* 275*	 SD 50 200 275* 275*
S4				 SD 150 175 175	 SD 50 125 200	 SD 50 150 200	 SD 50 175 200	 SD 50 200 200
S5				 SD 150 125 125	 SD 50 125 125	 SD 50 150 125	 SD 50 175 125	 SD 50 200 125
S6				 SD 150 150 150	 SD 50 125 125	 SD 50 150 150	 SD 50 175 175	 SD 50 200 200

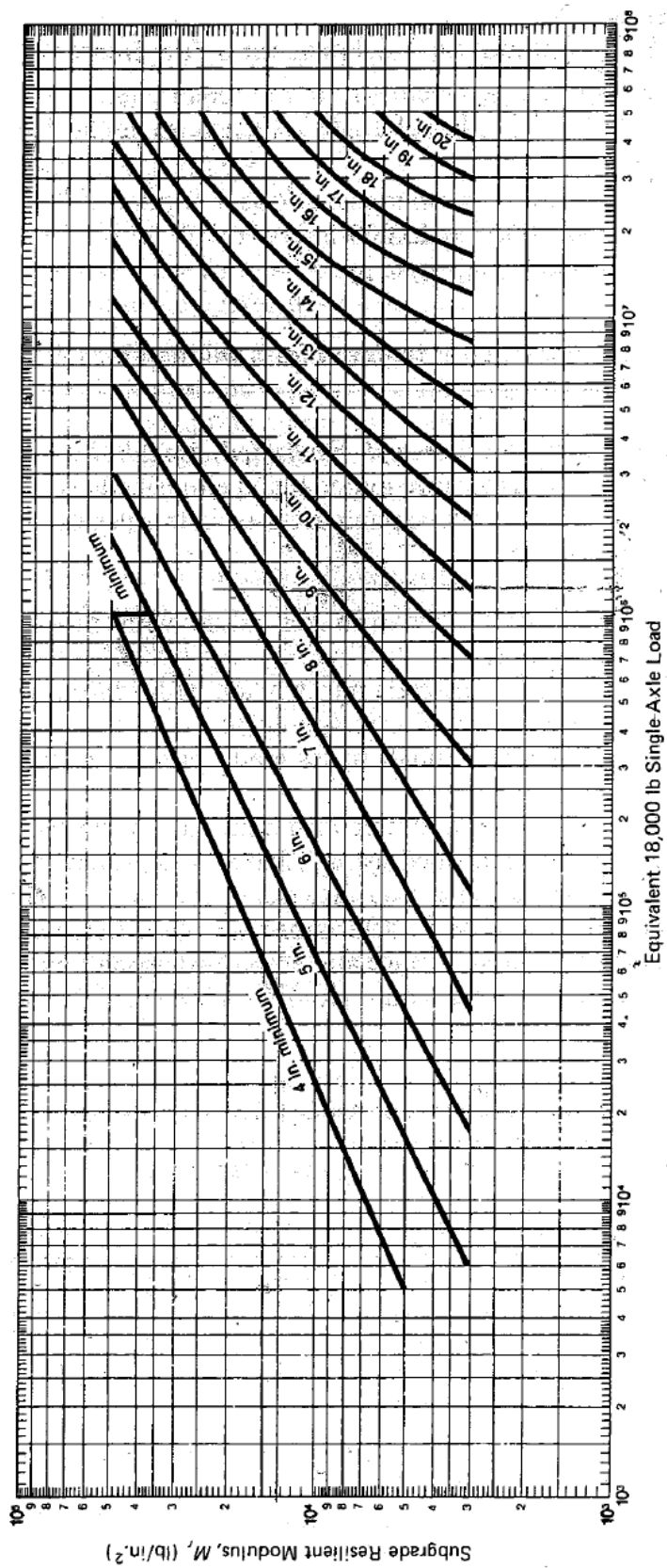
- Note: 1 * Up to 100mm of sub-base may be substituted with selected fill provided the sub-base is not reduced to less than the roadbase thickness or 200mm whichever is the greater. The substitution ratio of sub-base to selected fill is 25mm : 32mm.
- 2 A cement or lime-stabilised sub-base may also be used but see Section 7.7.2.

CHART 8 CEMENTED ROADBASE / SURFACE DRESSING

	T1	T2	T3	T4	T5	T6	T7	T8
S1								
S2								
S3								
S4								
S5								
S6								

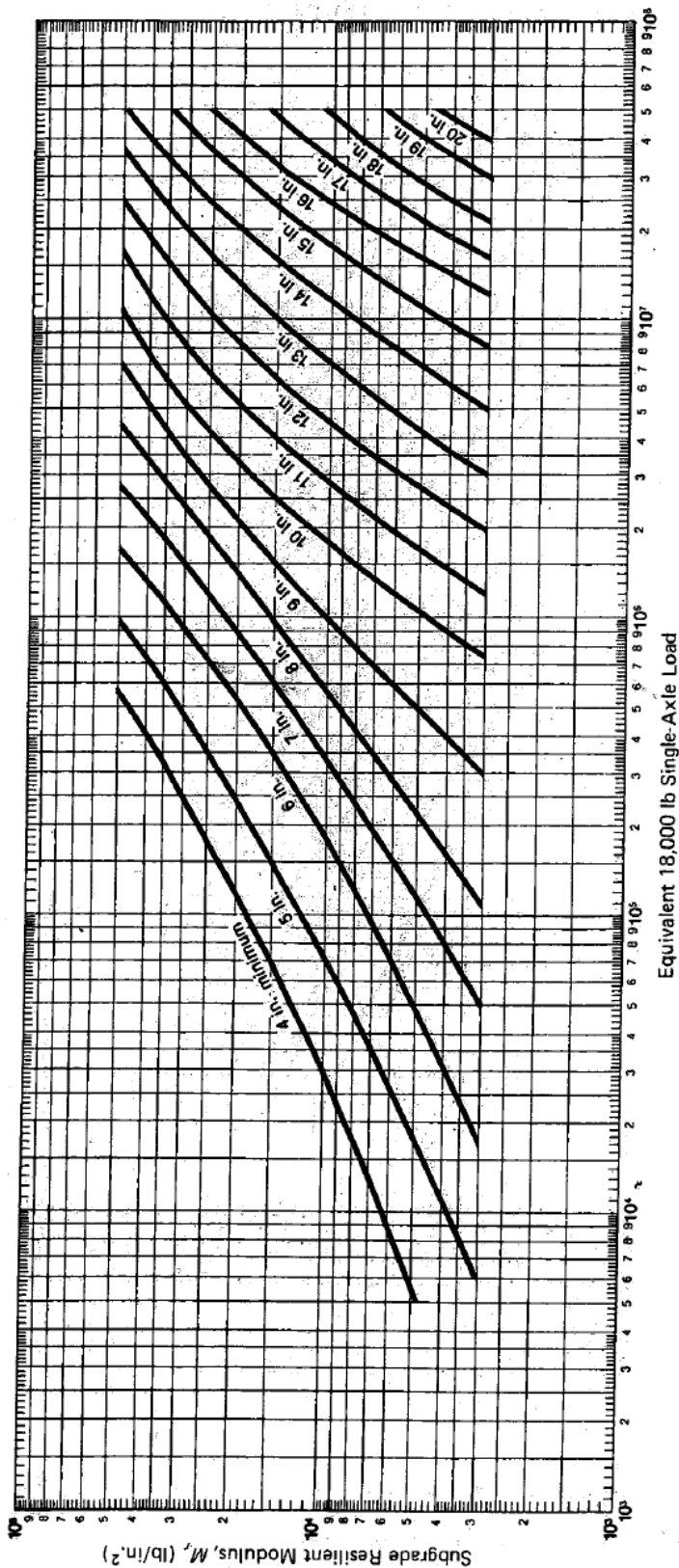
Note: A granular sub-base may also be used.

APPENDIX D: Asphalt Institute Method Design Charts



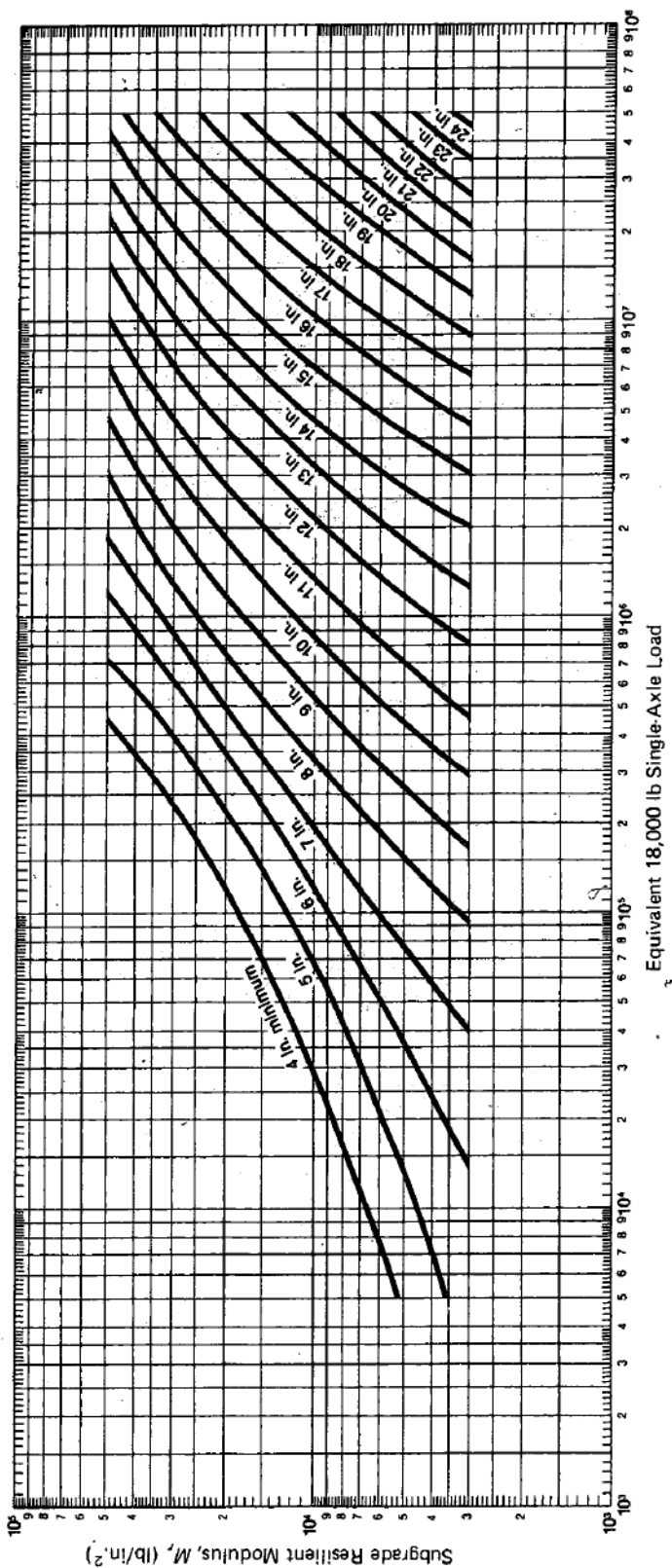
Source: Reproduced from *Thickness Design—Asphalt Pavements for Highways and Streets*, Manual Series No. 1, The Asphalt Institute, College Park, Md., September 1981.

Design Chart for Full-Depth Asphalt



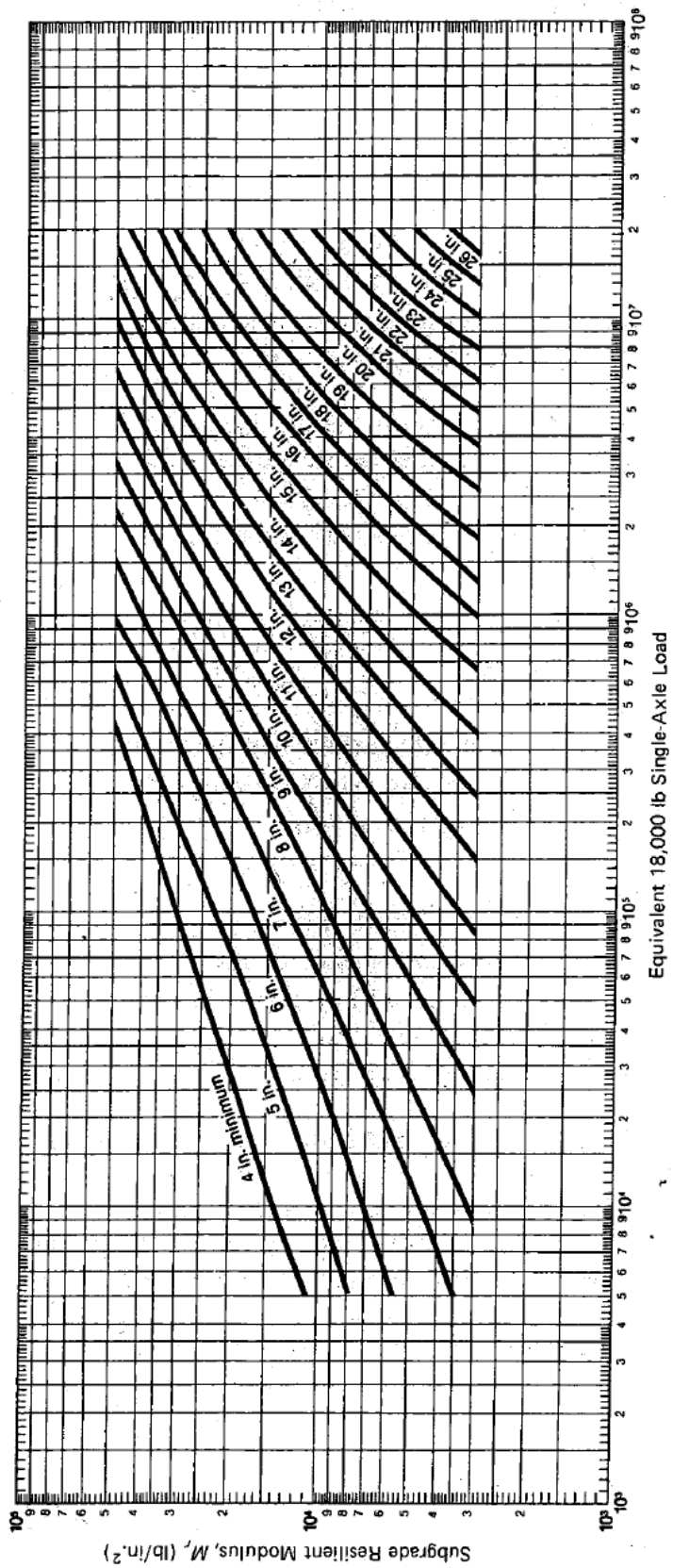
Source: Reproduced from *Thickness Design—Asphalt Pavements for Highways and Streets*, Manual Series No. 1, The Asphalt Institute, College Park, Md., September 1981.

Design Chart for Emulsified Asphalt Mix Type I



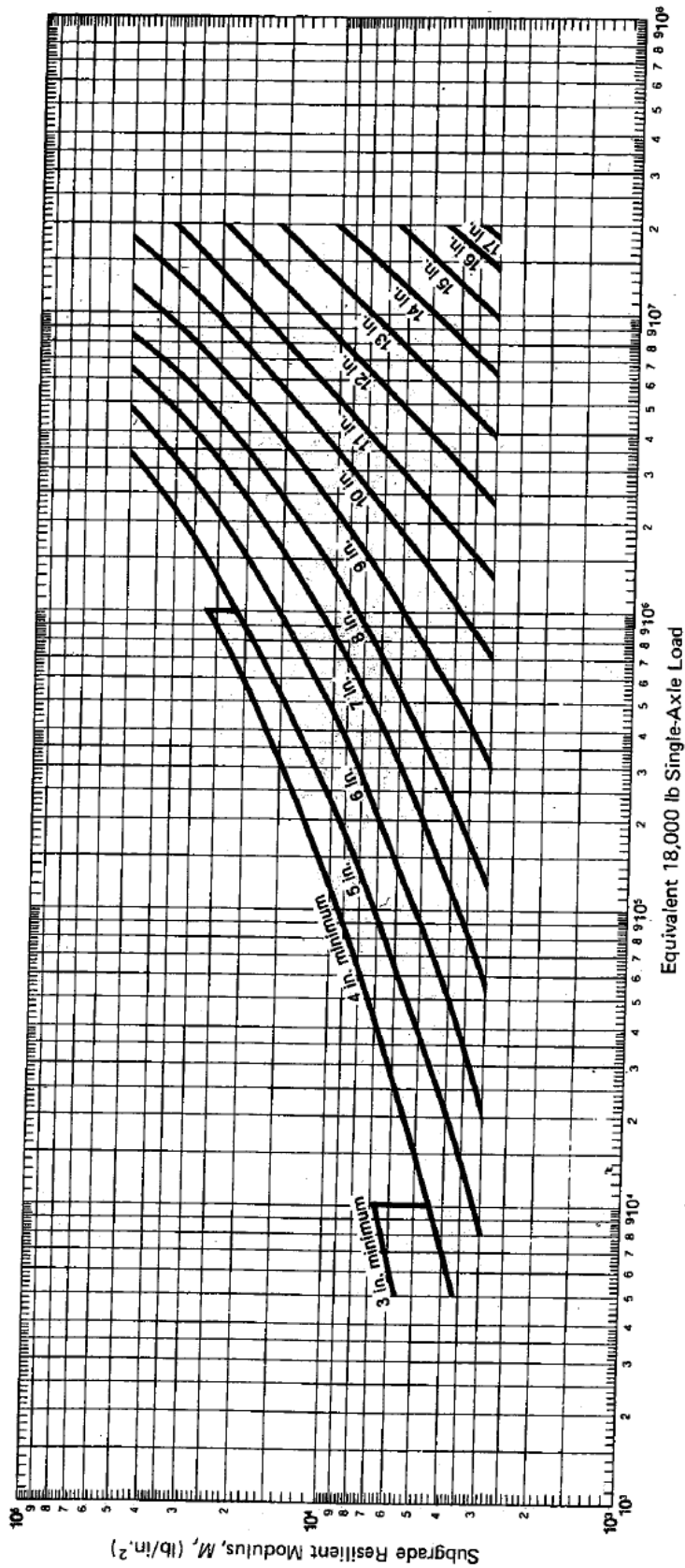
Source: Reproduced from *Thickness Design—Asphalt Pavements for Highways and Streets*, Manual Series No. 1, The Asphalt Institute, College Park, Md., September 1981.

Design Chart for Emulsified Asphalt Mix Type II



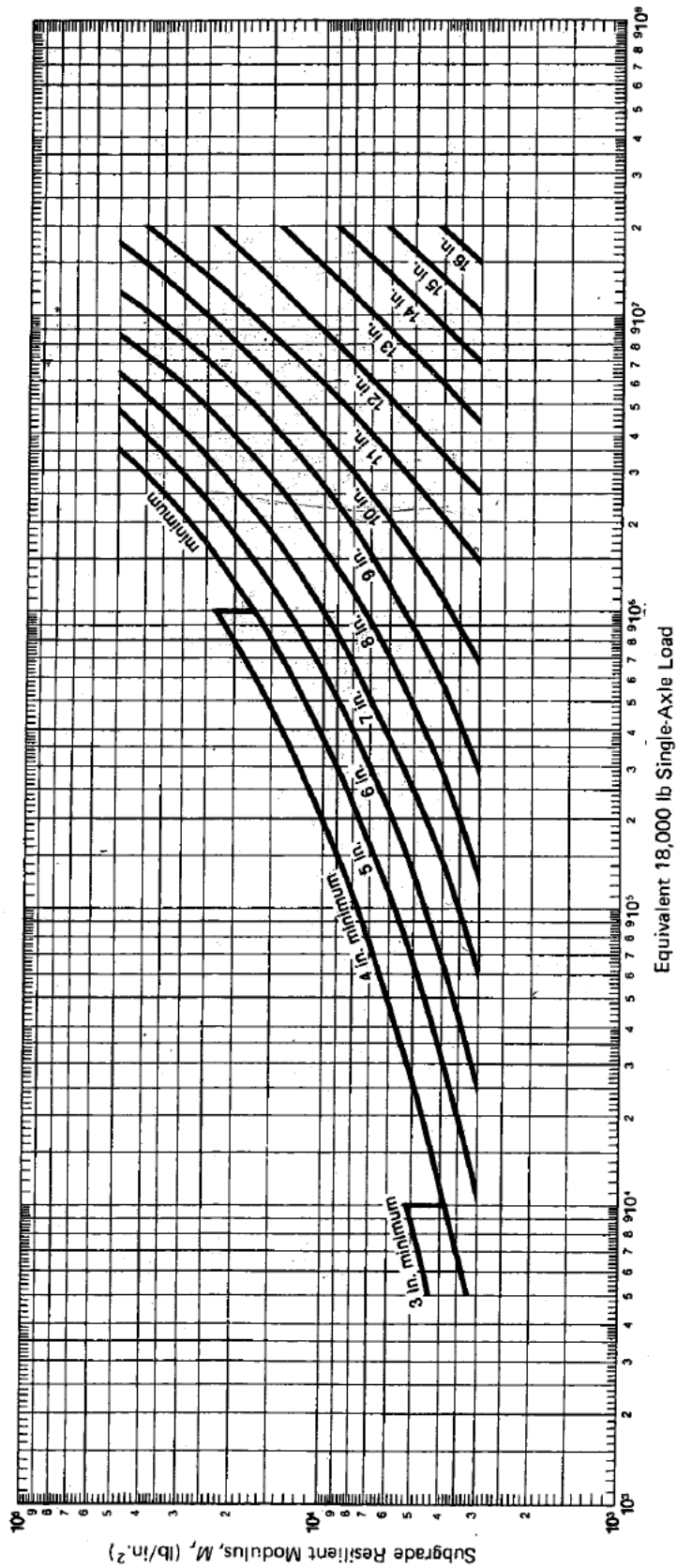
Source: Reproduced from *Thickness Design—Asphalt Pavements for Highways and Streets*, Manual Series No. 1, The Asphalt Institute, College Park, Md., September 1981.

Design Chart for Emulsified Asphalt Mix Type III



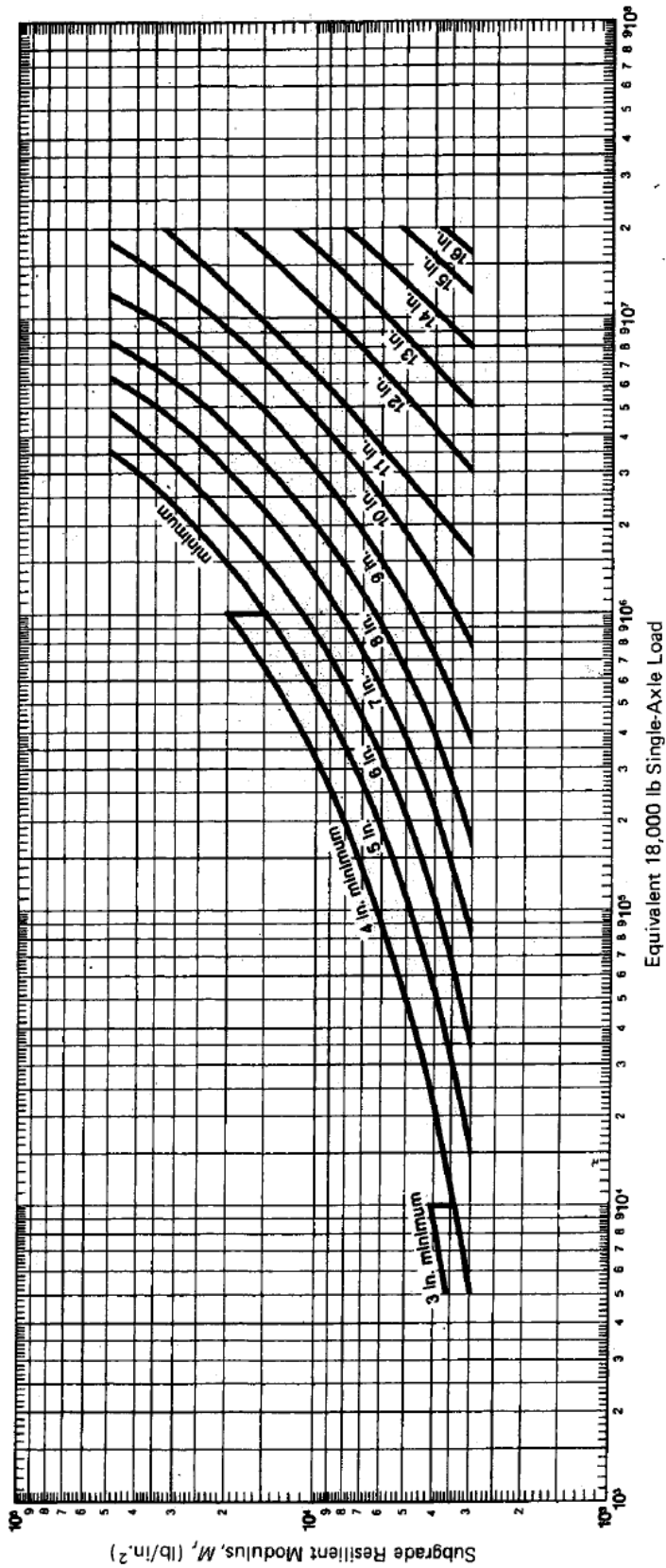
Source: Reproduced from *Thickness Design—Asphalt Pavements for Highways and Streets*, Manual Series No. 1, The Asphalt Institute, College Park, Md., September 1981.

Design Chart for Pavements
with Asphalt Concrete Surface
and Untreated Aggregate Base
100 mm thick



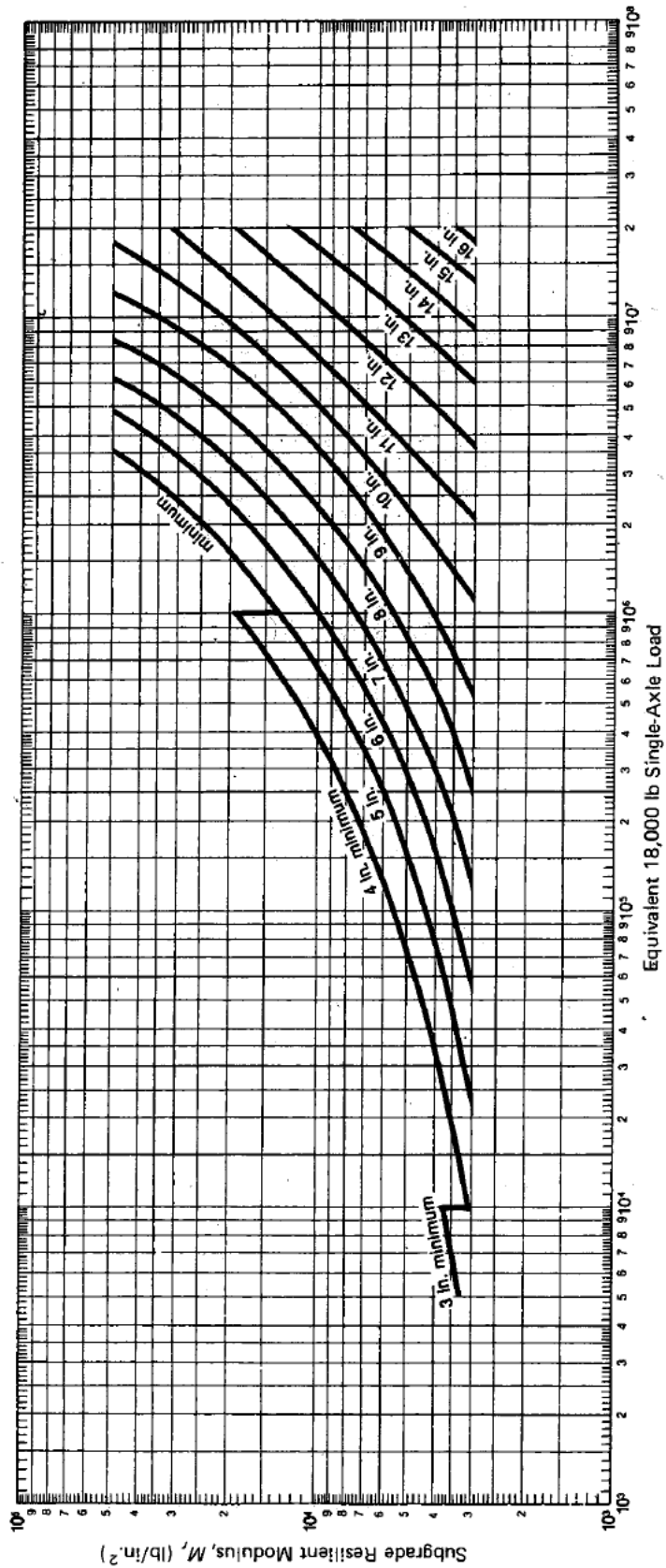
Source: Reproduced from *Thickness Design—Asphalt Pavements for Highways and Streets*, Manual Series No. 1, The Asphalt Institute, College Park, Md., September 1981.

Design Chart for Pavements
with Asphalt Concrete Surface
and Untreated Aggregate Base
150 mm thick



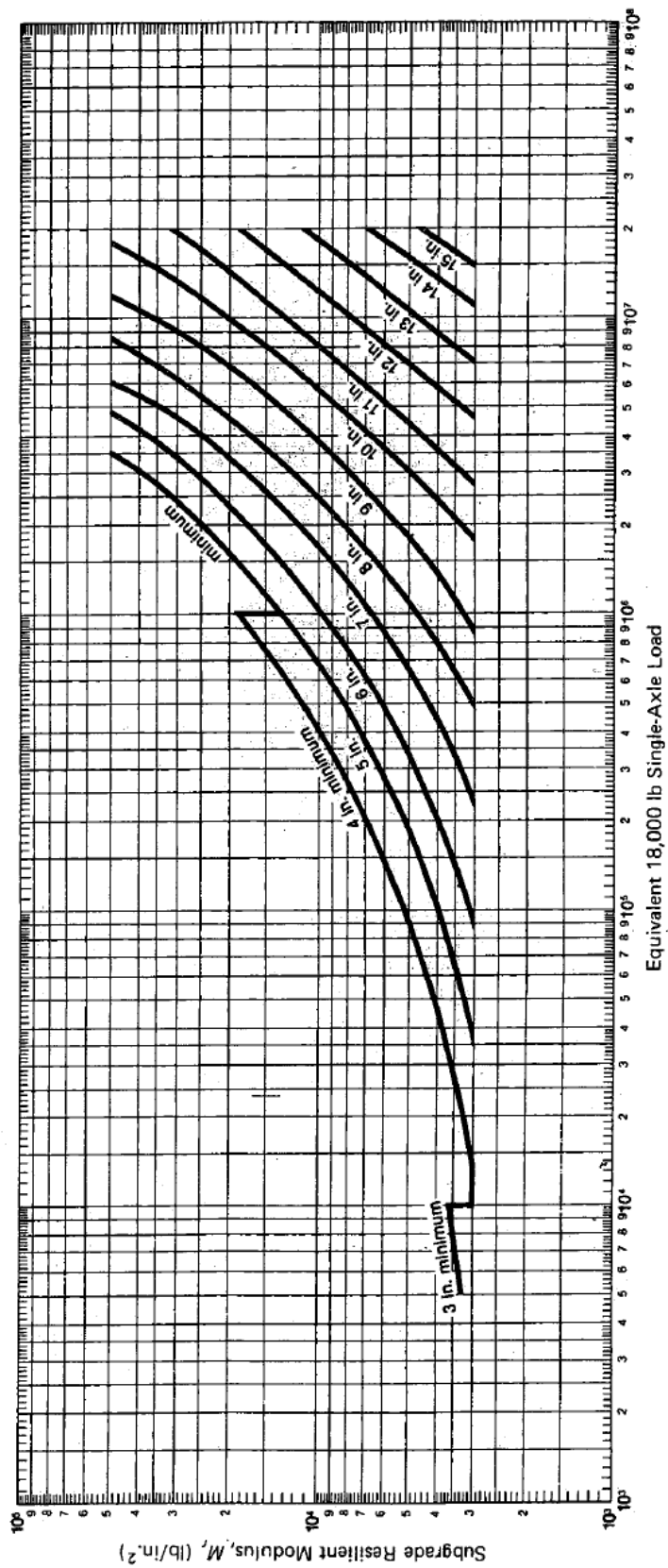
Source: Reproduced from *Thickness Design—Asphalt Pavements for Highways and Streets*, Manual Series No. 1, The Asphalt Institute, College Park, Md., September 1981.

Design Chart for Pavements
with Asphalt Concrete Surface
and Untreated Aggregate Base
200 mm thick



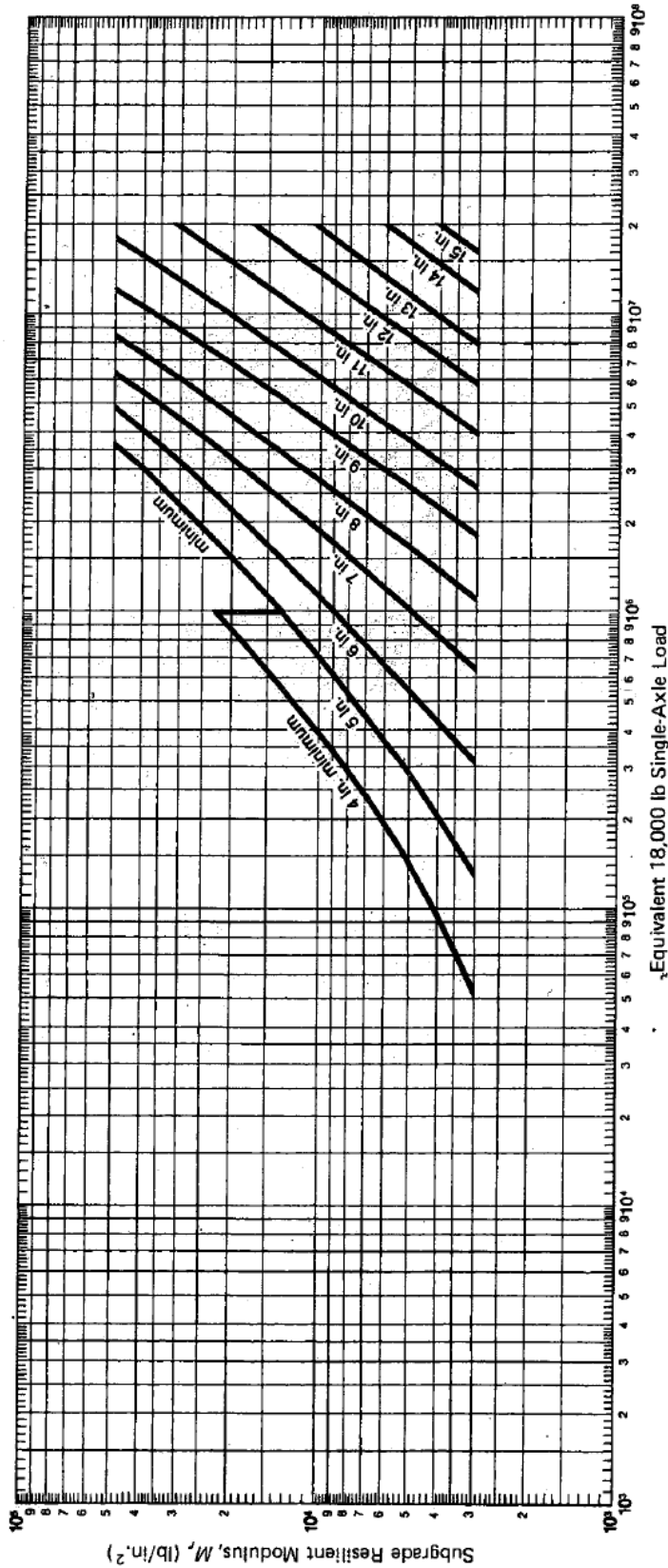
Source: Reproduced from *Thickness Design—Asphalt Pavements for Highways and Streets*, Manual Series No. 1, The Asphalt Institute, College Park, Md., September 1981.

Design Chart for Pavements
with Asphalt Concrete Surface
and Untreated Aggregate Base
250 mm thick



Source: Reproduced from *Thickness Design—Asphalt Pavements for Highways and Streets*, Manual Series No. 1, The Asphalt Institute, College Park, Md., September 1981.

Design Chart for Pavements
with Asphalt Concrete Surface
and Untreated Aggregate Base
300 mm thick



Source: Reproduced from *Thickness Design—Asphalt Pavements for Highways and Streets*, Manual Series No. 1, The Asphalt Institute, College Park, Md., September 1981.

Design Chart for Pavements
with Asphalt Concrete Surface
and Untreated Aggregate Base
450 mm thick