FEDERAL DEMOCRATIC REPUBLIC OF ETHIOPIA

ETHIOPIAN ROADS AUTHORITY

PAVEMENT DESIGN MANUAL

VOLUME I
FLEXIBLE PAVEMENTS AND GRAVEL ROADS

2002

PAVEMENT DESIGN MANUAL

VOLUME I FLEXIBLE PAVEMENTS AND GRAVEL ROADS -2002

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PREFACE

This Pavement Design Manual - 2002 (Volume 1: Flexible Pavements and Gravel Roads) is one of the series of "Design Manuals, Standard Contract Documents and Specifications" prepared under a credit financing of the International Development Agency (IDA). The consulting services were provided by the Louis Berger Group, Inc.

This manual has been developed from current international practice appropriately modified to take account of local experience and conditions. It is written for the practicing engineer.

ERA formed a Project Working Group charged with evaluating and commenting upon the draft Manuals and guiding the Consultant on the preparation of the final Manuals. Members of the Working Group and the Louis Berger Group team for this Manual include the following:

1. Ato Belaynehe Wolde Selassie
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Companion Documents and Manuals Prepared under the same service are the following:

1. Geometric Design Manual - 2002
2. Site Investigation Manual - 2002
3. Pavement Design Manual Volume II - 2002 (Rigid Pavements)
4. Overlay/Rehabilitation Manual - 2002
5. Drainage Design Manual Volume I and II - 2002
6. Bridge Design Manual - 2002
7. Standard Environmental Methodologies and Procedures Manual - 2002
8. Standard Technical Specifications - 2002
9. Standard Detail Drawings - 2002
10. Standard Bidding Documents For Road Work Contracts
    National Competitive Bidding (NCB) - 2002
11. Standard Tendering Documents For Road Work Contracts
    International Competitive Bidding (ICB) - 2002

Appropriate reviews and comments were also provided by agencies and individuals through ERA's Project Working Group. The Working Group wishes to acknowledge the contributions made from all other specialists within and outside of ERA in the preparation of these Manuals and Documents.

The layout of this Manual has been arranged with the following hierarchy:

- Chapter
- Major heading or Section (level one)
- Sub-Section within the major heading (level two)
- Sub-sub-Section within the second level subject matter (level three)

All tables are described by number beginning with one (1) at the first of each Chapter. Figures are described in a similar manner.

Updates:

This manual will be updated and revised from time to time, as deemed appropriate. Significant changes to criteria, procedures or any other relevant issues related to the new policies or revised law of the land, ERA, or that is mandated by the relevant Federal Government Ministry or Agency, should be amended and incorporated in the manual as soon as possible after their date of effectiveness.
Other minor changes, not affecting the whole nature of this manual, may be accumulated and made periodically. When a change is approved, new page(s) instituting the revisions, together with the revision date, will be issued and inserted in to the relevant Chapters.

All revisions to the Pavement Design Manual – Volume I will be made strictly in accordance with the following procedures:

(1) Any proposed change will be submitted by or through the Head of the Design (Branch, Division)of ERA.

(2) The proposed change, addition, or deletion will be submitted on a Manual Change form 1-1(see the attached form) and forwarded with an explanation of its need and purpose.

(3) If the change is approved, the General Manager will sign the Manual Change form and return a copy to the Head of the Design (Branch, Division), who will arrange for the change to be incorporated into the Geometric Design Manual.

(4) The Head of the Design (Branch, Division) will re-issue all effected pages of the manual showing the proper revision date as shown on the Manual Change form.1-1.

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Submitted by: __________________ Approved by: ____________________________

Head of Design (*Branch*, *Division* **)** ERA General Manager

Date: __________________ **Date:** __________________

Manual change Form 1-1

FOREWORD

This is the first comprehensive Pavement Design Manual for Flexible Pavements and Gravel Roads prepared for the use and technical guidance of design personnel of the Ethiopian Roads Authority and consultants doing Pavement Design work for the Authority. However, it may also be used as a guide by other agencies undertaking relevant work in the road sector.

The Ethiopian Roads Authority has prepared this Manual under a credit from the International Development Agency (IDA) for design of roads in order to standardize design practices in all ERA design works.

The road network comprises a huge national asset that requires adherence to appropriate standards for design, construction and maintenance in order to provide a high level of service. As the length of the engineered road network is increasing, appropriate choice of methods to preserve this investment becomes increasingly important.

This Manual has particular reference to the prevailing conditions in Ethiopia and reflects ERA's experience gained through activities within the road sector during the last 50 years.

The design standards set out in this Manual shall be adhered to unless otherwise directed by the concerned bodies with in ERA. However, I will like to emphasize that careful consideration to sound engineering practice shall be observed in the use of the Manual, and under no circumstances shall the Manual waive professional judgment in applied engineering. For simplification in reference this Manual may be cited as "ERA Pavement Design Manual Volume I–2002."

It is my sincere hope that this Manual will provide all users with both a standard reference and a ready source of good practice for the Flexible Pavement and Gravel Road Pavement design, and will assist in a cost effective operation, and environmentally sustainable development of our road network.

I look forward to the practices contained in this Manual being quickly adopted into our operations, thereby making a sustainable contribution to the improved infrastructure of our country.

As this Manual due to technological development and change, requires periodic updating, comments and suggestions on all aspects from any concerned body, group or individual as feedback during its implementation is expected and will be highly appreciated.

Tesfamichael Nahusenay

General Manager

Ethiopian Roads Authority

ACKNOWLEDGEMENTS

This Pavement Design Manual Volume 1 Flexible Pavements and Gravel Roads-2002 is based on a review of the design standards of several countries. Most chapters are based closely on the Transport Research Laboratory Overseas Road Note 31: A Guide to the Structural Design of Bitumen-Surfaced Roads in Tropical and Sub-Tropical Countries (ref.1). Of note is the fact that this reference document and companion TRL documents have drawn on the experience of TRL and collaborating organizations in several tropical and sub-tropical countries, including Ethiopia.

For that portion of the text dealing with surface treatments, we have based our text once again on a TRL source, Overseas Road Note 3: A Guide to Surface Dressing in Tropical and Sub-Tropical Countries (ref. 2).

For that portion of the text dealing with gravel roads, of particular use was TRRL Research Report 147 (1998), The Performance of Experimental Weathered Basalt Gravel Roads in Ethiopia (ref. 3). Other references included TRRL's Experimental Use of Cinder Gravels on Roads in Ethiopia, 1987 (ref. 4), and Newill and Kassaye's The Location and Engineering Properties of Volcanic Cinders Gravels in Ethiopia, 1980 (ref. 5).

Other major reference sources include AASHTO, and, in particular, the AASHTO Guide for Design of Pavement Structures, as revised in 1993 (ref.6). References of the Asphalt Institute were reviewed for asphalt concrete and other hot-mix types. South African and Kenyan references were reviewed to assist in the development of a design well suited for the eastern African region in general and Ethiopia in particular.

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

The authors of the manual are listed in the preface.

A limited number of copies of this manual are available to other organizations and to individuals with an interest in road pavement design. Inquiries should be made to:

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Ethiopian Roads Authority
Post Office Box 1770
Addis Ababa, Ethiopia

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DEFINITIONS

Aggregate	Hard mineral elements of construction material mixtures, for example: sand, gravel (crushed or uncrushed) or crushed rock.
Asphalt Concrete	A mixture to predetermined proportions of aggregate, filler and bituminous binder material plant mix and usually placed by means of a paving machine.
Asphalt Surfacing	The layer or layers of asphalt concrete constructed on top of the roadbase, and, in some cases, the shoulders.
Average Annual Daily Traffic (AADT)	The total yearly traffic volume in both directions divided by the number of days in the year.
Average Daily Traffic (ADT)	The total traffic volume during a given time period in whole days greater than one day and less than one year divided by the number of days in that time period.
Base Course	This is the main component of the pavement contributing to the spreading of the traffic loads. In many cases, it will consist of crushed stone or gravel, or of good quality gravelly soils or decomposed rock. Bituminous base courses may also be used (for higher classes of traffic). Materials stabilized with cement or lime may also be contemplated.
Binder Course	The lower course of an asphalt surfacing laid in more than one course.
Borrow Area	An area within designated boundaries, approved for the purpose of obtaining borrow material. A borrow pit is the excavated pit in a borrow area.
Borrow Material	Any gravel, sand, soil, rock or ash obtained from borrow areas, dumps or sources other than cut within the road prism and which is used in the construction of the specified work for a project. It does not include crushed stone or sand obtained from commercial sources.
Boulder	A rock fragment, usually rounded by weathering or abrasion, with an average dimension of 0.30 m or more.
Bound Pavement Materials	Pavement materials held together by an adhesive bound between the materials and another binding material such as bitumen.
Camber	The convexity given to the curved cross-section of a roadway.
Capping Layer	(selected or improved subgrade). The top of embankment or bottom of excavation prior to construction of the pavement structure. Where very weak soils and/or expansive soils (such as black cotton soils) are encountered, a capping layer is sometimes necessary. This consists of better quality subgrade material imported from elsewhere or subgrade material improved by stabilization (usually mechanical), and may also be considered as a lower quality subbase.
Carriageway	That portion of the roadway including the various traffic lanes and auxiliary lanes but excluding shoulders.

Cross-Section	A vertical section showing the elevation of the existing ground, ground data and recommended works, usually at right angles to the centerline.
Crossfall	The difference in level measured transversely across the surface of the roadway.
Culvert	A structure, other than a bridge, which provides an opening under the carriageway or median for drainage or other purposes.
Cutting	Cutting shall mean all excavations from the road prism including side drains, and excavations for intersecting roads including, where classified as cut, excavations for open drains.
Chippings	Stones used for surface dressing (treatment).
Design Period	The period of time that an initially constructed or rehabilitated pavement structure will perform before reaching a level of deterioration requiring more than routine or periodic maintenance.
Diverted Traffic	Traffic that changes from another route (or mode of transport) to the project road because of the improved pavement, but still travels between the same origin and destination.
Equivalency Factors	Used to convert traffic volumes into cumulative standard axle loads.
Equivalent Single Axle Load (ESA)	Summation of equivalent 8.16 ton single axle loads used to combine mixed traffic to design traffic for the design period.
Fill	Material of which a man-made raised structure or deposit such as an embankment is composed, including soil, soil-aggregate or rock. Material imported to replace unsuitable roadbed material is also classified as fill.
Flexible Pavements	Includes primarily those pavements that have a bituminous (surface dressing or asphalt concrete) surface. The terms "flexible and rigid" are somewhat arbitrary and were primarily established to differentiate between asphalt and Portland cement concrete pavements.
Formation Level	Level at top of subgrade.
Generated Traffic	Additional traffic which occurs in response to the provision of improvement of the road.
Grading Modulus (GM)	The cumulative percentages by mass of material in a representative sample of aggregate, gravel or soil retained on the 2.00 mm, 0.425 mm and 0.075 mm sieves, divided by 100.
Heavy Vehicles	Those having an unloaded weight of 3000 kg or more.
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.
Mountainous (Terrain)	Terrain that is rugged and very hilly with substantial restrictions in both (terrain) horizontal and vertical alignment.

Normal Traffic	Traffic which would pass along the existing road or track even if no new pavement were provided.
Overlay	One or more courses of asphalt construction on an existing pavement. The overlay often includes a leveling course, to correct the contour of the old pavement, followed by a uniform course or courses to provide needed thickness.
Pavement Layers	The layers of different materials which comprise the pavement structure.
Project Specifications	The specifications relating to a specific project, which form part of the contract documents for such project, and which contain supplementary and/or amending specifications to the standard specifications
Quarry	An area within designated boundaries, approved for the purpose of obtaining rock by sawing or blasting.
Reconstruction	The process by which a new pavement is constructed, utilizing mostly new materials, to replace an existing pavement.
Rehabilitation	Work undertaken to significantly extend the service life of an existing pavement. This may include overlays and preoverlay repairs, and may include complete removal and reconstruction of the existing pavement, or recycling of part of the existing materials.
Roadbase	A layer of material of defined thickness and width constructed on top of the subbase, or in the absence thereof, the subgrade. A roadbase may extend to outside the carriageway.
Road Bed	The natural in situ material on which the fill, or in the absence of fill, any pavement layers, are to be constructed.
Road Bed Material	The material below the subgrade extending to such depth as affects the support of the pavement structure.
Road Prism	That portion of the road construction included between the original ground level and the outer lines of the slopes of cuts, fills, side fills and side drains. It does not include subbase, roadbase, surfacing, shoulders, or existing original ground.
Roadway	The area normally traveled by vehicles and consisting of one or a number of contiguous traffic lanes, including auxiliary lanes and shoulders.
Rolling (Terrain)	Terrain with low hills introducing moderate levels of rise and fall with some restrictions on vertical alignment.
Side Fill	That portion of the imported material within the road prism which lies outside the fills, shoulders, roadbase and subbase and is contained within such surface slopes as shown on the Drawings or as directed by the Engineer. A distinction between fills and side fill is only to be made if specified.
Side Drain	Open longitudinal drain situated adjacent to and at the bottom of cut or fill slopes.
Stabilization	The treatment of the materials used in the construction of the road bed material, fill or pavement layers by the addition of a cementitious binder such as lime or Portland Cement or the mechanical modification of the

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

Subbase The layer of material of specified dimensions on top of the subgrade and below the roadbase. It is the secondary load-spreading layer underlying the base course. It will usually consist of a material of lower quality than that used in the base course and particularly of lower bearing strength. Materials may be unprocessed natural gravel, gravel-sand, or gravel-sand-clay, with controlled gradation and plasticity characteristics. The subbase also serves as a separating layer preventing contamination of the base course by the subgrade material and may play a role in the internal drainage of the pavement.

Subgrade The surface upon which the pavement structure and shoulders are constructed. It is the top portion of the natural soil, either undisturbed (but recompacted) local material in cut sections, or soil excavated in cut or borrow areas and placed as compacted embankment.

Subsurface Drain Covered drain constructed to intercept and remove subsoil water, including any pipes and permeable material in the drains.

Surface Treatment The sealing or resealing of the carriageway or shoulders by means of one or more successive applications of bituminous binder and crushed stone chippings.

Surfacing This comprises the top layers(s) of the flexible pavement and consists of a bituminous surface dressing or one or two layers of premixed bituminous material (generally asphalt concrete). Where premixed materials are laid in two layers, these are known as the wearing course and the binder course as shown in Figure 1-2.

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

Traffic Volume Volume of traffic usually expressed in terms of average annual daily traffic (AADT).

Typical Cross-Section A cross-section of a road showing standard dimensional details and features of construction.

Unbound Pavement Materials Naturally occurring or processed granular material which is not held together by the addition of a binder such as cement, lime or bitumen.

Wearing Course The top course of an asphalt surfacing or, for gravel roads, the uppermost layer of construction of the roadway made of specified materials.

ABBREVIATIONS

AADT	Average Annual Daily Traffic
ADT	Average Daily Traffic
AC	Asphalt concrete
AASHO	American Association of State Highway Officials (previous designation)
AASHTO	American Association of State Highway and Transportation Officials
BS	British Standard
CBR	California Bearing Ratio (as described in AASHTO T 193)
ERA	Ethiopian Roads Authority
ESA	Equivalent Single Axle
ICL	Initial Consumption of Lime Test
MDD	Maximum Dry Density
NDT	Non-Destructive Testing
PMS	Pavement Management System
TRRL	Transport and Road Research Laboratory (UK)
TRL	Transport Research Laboratory (UK)
VMA	Voids in mineral aggregate

1. INTRODUCTION

1.1 General

This manual gives recommendations for the structural design of flexible pavement and gravel roads in Ethiopia. The manual is intended for engineers responsible for the design of new road pavements and is appropriate for roads which are required to carry up to 30 million cumulative equivalent standard axles in one direction. This upper limit is suitable at present for the most trafficked roads in Ethiopia.

The design of strengthening overlays for existing bitumen-surfaced roads is not covered in this manual, as this is the object of the ERA *Overlay and Rehabilitation Manual-2002.* Similarly, the design of cement concrete surfaced roads is treated separately in the ERA *Pavement Design Manual Volume II-2002 (Rigid Pavements).*

1.2 Underlying Principles

1.2.1 Flexible Pavements

Road flexible pavements are intended to limit the stress created at the subgrade level by the traffic traveling on the pavement surface, so that the subgrade is not subject to significant deformations. In effect, the concentrated loads of the vehicle wheels are spread over a sufficiently larger area at subgrade level. At the same time, the pavement materials themselves should not deteriorate to such an extent as to affect the riding quality and functionality of the pavement. These goals must be achieved throughout a specific design period.

Pavements do deteriorate, however, due to time, climate and traffic. Therefore, the goal of the pavement design is to limit, during the period considered, deteriorations which affect the riding quality, such as, in the case of flexible pavements, cracking, rutting, potholes and other such surface distresses to acceptable levels.

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

It is understandable that what constitutes an “acceptable riding quality” depends on what the users expect. For roads with higher traffic, higher geometric standards, and higher vehicle speeds as a consequence, less distress will be expected and considered acceptable. Hence, for instance, trunk and link roads may be expected to offer some higher rideability than access, collector roads, etc. in a similar design period. Similarly, gravel roads may be expected to offer a lower riding quality. These differences are implicitly considered in the design, although in broad terms rather than in precise measurable economic terms.

1.2.2 Gravel Roads

Unpaved roads consist of gravel wearing courses. Gravel pavements are also designed to a minimum thickness required to avoid excessive strain at the subgrade level. This in turn ensures that the subgrade is not subject to significant deformations. At the same time, the gravel materials themselves should not deteriorate to such an extent as to affect the riding quality and functionality of the pavement. These goals must be achieved throughout a specific design period. Deteriorations which affect the riding quality of a gravel road include rutting, potholes, corrugations, and other such distresses.

Gravel wearing courses must also be designed for an additional thickness to compensate for gravel loss under traffic during the period between regravelling operations. Such thicknesses are dependent on the subgrade strength class and the traffic class.

1.3 Overview of Pavement Structures

1.3.1 General

The basic idea in building a pavement for all-weather use by vehicles is to prepare a suitable subgrade, provide necessary drainage and construct a pavement that will:

- Have sufficient total thickness and internal strength to carry expected traffic loads;
- Have adequate properties to prevent or minimize the penetration or internal accumulation of moisture, and
- Have a surface that is reasonably smooth and skid resistant at the same time, as well as reasonably resistant to wear, distortion and deterioration by weather.

The subgrade ultimately carries all traffic loads. Therefore, the structural **function** of a pavement is to support a wheel load on the pavement surface, and transfer and spread that load to the subgrade without exceeding either the strength of the subgrade or the internal strength of the pavement itself.

Figure 1-1 shows wheel load, W, being transmitted to the pavement surface through the tire at an approximately uniform vertical pressure, P_o. The pavement then spreads the wheel load to the subgrade so that the maximum pressure on the subgrade is only P_1. By proper selection of pavement materials and with adequate pavement thickness, P_1 will be small enough to be easily supported by the subgrade. In its simple form, Figure 1-1 illustrates a principle valid for the various pavement types discussed below, albeit with variations in the magnitude and mechanism of stress distribution.

1.3.2 Pavement Types

The elements of a flexible pavement are illustrated in Figure 1-2, where the simpler form of a pavement provided by the wearing course of a gravel road is also shown.

The classical definition of flexible pavements primarily includes those pavements that have a bituminous (surface dressing or asphalt concrete) surface. By contrast, the classical rigid (or concrete) pavement is made up of Portland cement concrete. The terms flexible and rigid are somewhat arbitrary and were primarily established to differentiate between asphalt and Portland cement concrete pavements.

The essential difference between the two types of pavements is the manner in which they distribute the load over the subgrade. The rigid pavement, because of its rigidity and high modulus of elasticity, tends to distribute the load over a relatively wide area of soil; thus, the slab itself supplies a major portion of the structural capacity. The major factor considered in the design of rigid pavements is the structural strength of the concrete, and a certain amount of variation in subgrade strength has little influence upon the structural capacity of the pavement. This volume of the Pavement Design Manual is devoted to flexible pavements and gravel roads. Designers should refer to Volume 2 for rigid pavements.

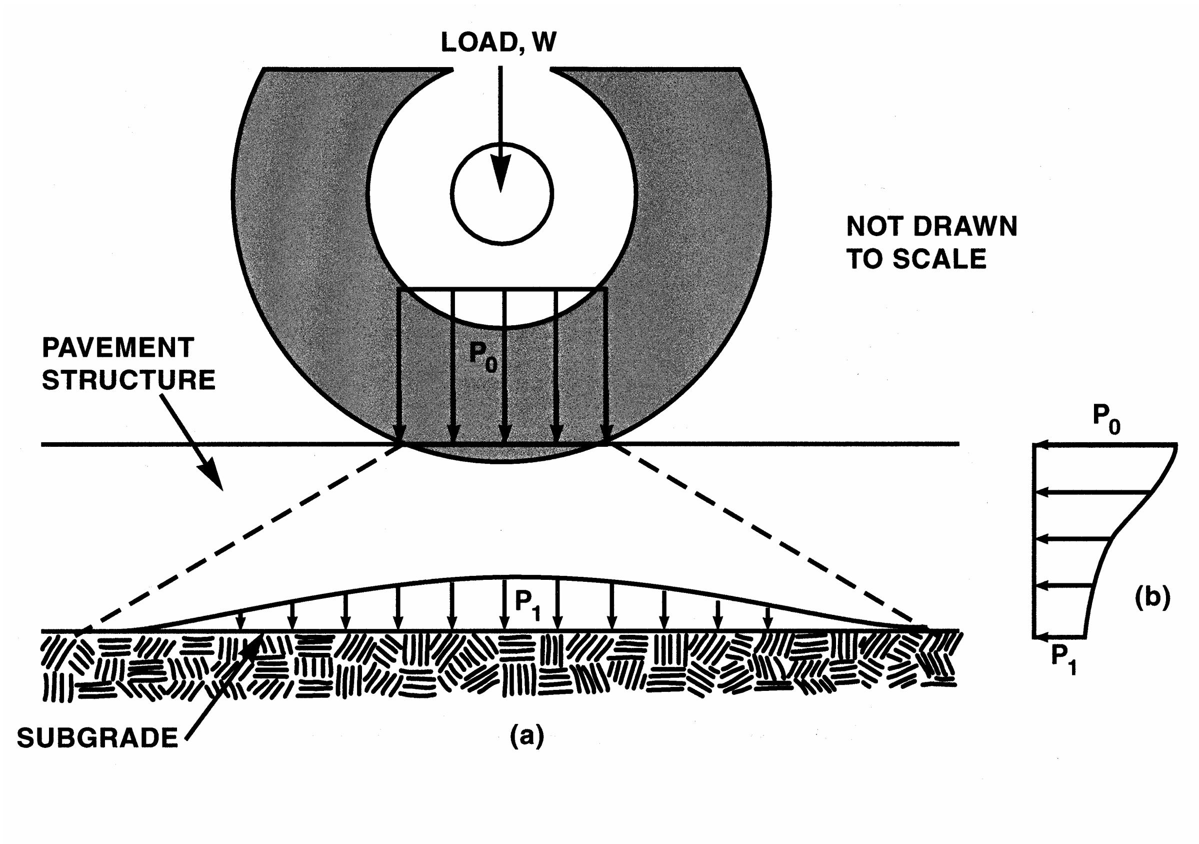


Figure 1-1: Spread of Wheel-Load through Pavement Structure

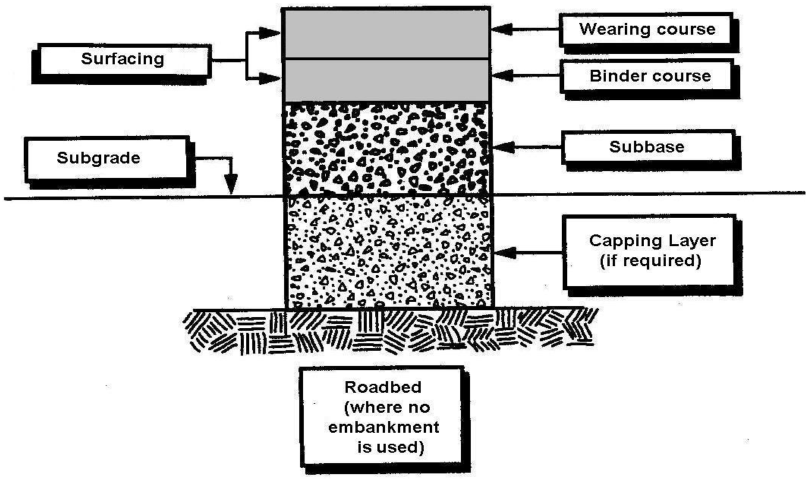


Figure 1-2: Elements of a Flexible Pavement

1.3.3 Flexible Pavements

To give satisfactory service, a flexible pavement must satisfy a number of structural criteria or considerations; some of these are illustrated in Figure 1-3. Some of the important considerations are:

(1) the subgrade should be able to sustain traffic loading without excessive deformation; this is controlled by the vertical compressive stress or strain at this level,
(2) bituminous materials and cement-bound materials used in roadbase design should not crack under the influence of traffic; this is controlled by the horizontal tensile stress or strain at the bottom of the roadbase,
(3) the roadbase is often considered the main structural layer of the pavement, required to distribute the applied traffic loading so that the underlying materials are not overstressed. It must be able to sustain the stress and strain generated within itself without excessive or rapid deterioration of any kind.
(4) in pavements containing a considerable thickness of bituminous materials, the internal deformation of these materials must be limited; their deformation is a function of their creep characteristics,
(5) the load spreading ability of granular subbase and capping layers must be adequate to provide a satisfactory construction platform.

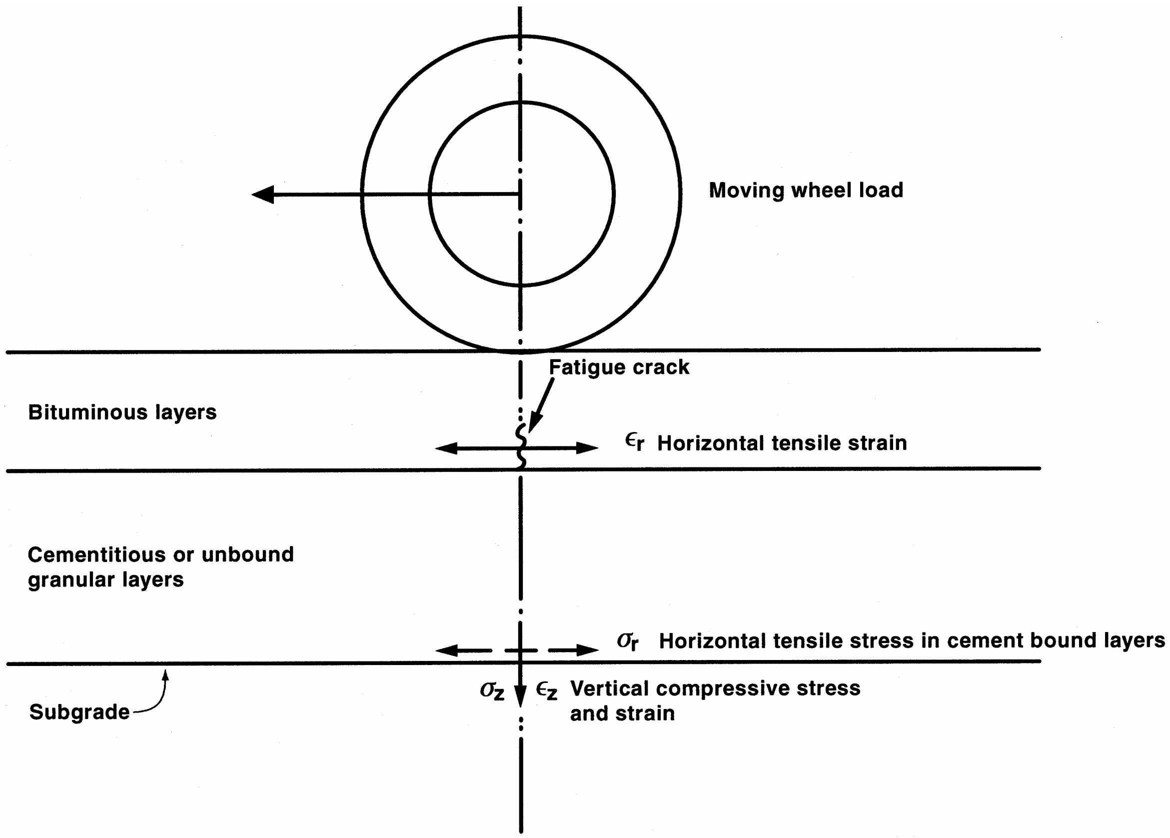


Figure 1-3: Critical Stresses and Strains in a Flexible Pavement

In practice, other factors have to be considered such as the effects of drainage.

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

1.3.4 Gravel Roads

Gravel roads represent the other type of design considered in this volume of the manual. The elements of a flexible pavement are illustrated in Figure 1-4, where the simpler form of a pavement provided by the wearing course of a gravel road is also shown.

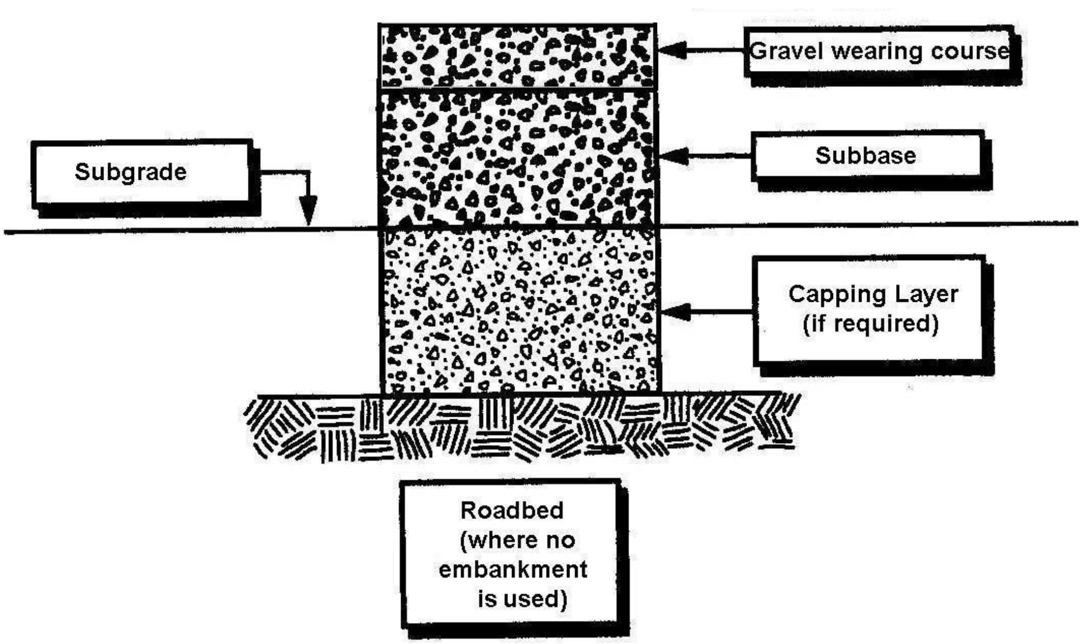


Figure 1-4: Elements of a Gravel Pavement

The mechanisms of deterioration of gravel roads differ from those of flexible pavement. While the functions of the wearing course still include the protection of the subgrade, and the wearing course needs to be designed for that aspect, the potential defects of a gravel road require other considerations in the design.

Typical defects which may affect gravel roads are dustiness, potholes, stoniness, corrugations, ruts, cracks, ravelling (formation of loose material), erosion, slipperiness, impassibility and loss of wearing course material. Many of these have a direct effect on the road roughness and safety.

Since corrugations are one of the most disturbing defects of gravel roads (and one which still causes much debate), an illustration of the likely mechanism of their formation is worthwhile, and is given in Figure 1-5. In illustration a), localized areas of the gravel wearing course have slightly lesser cohesion than adjacent areas, and a result is that the wheel displaces this material towards the back, at the same time compressing the remaining material at the contact point. Continuing actions as in a) result eventually in the wheel loosing contact with the road, as in b). When the wheel regains road contact, as in c), the result is a magnification of the effects as in a).

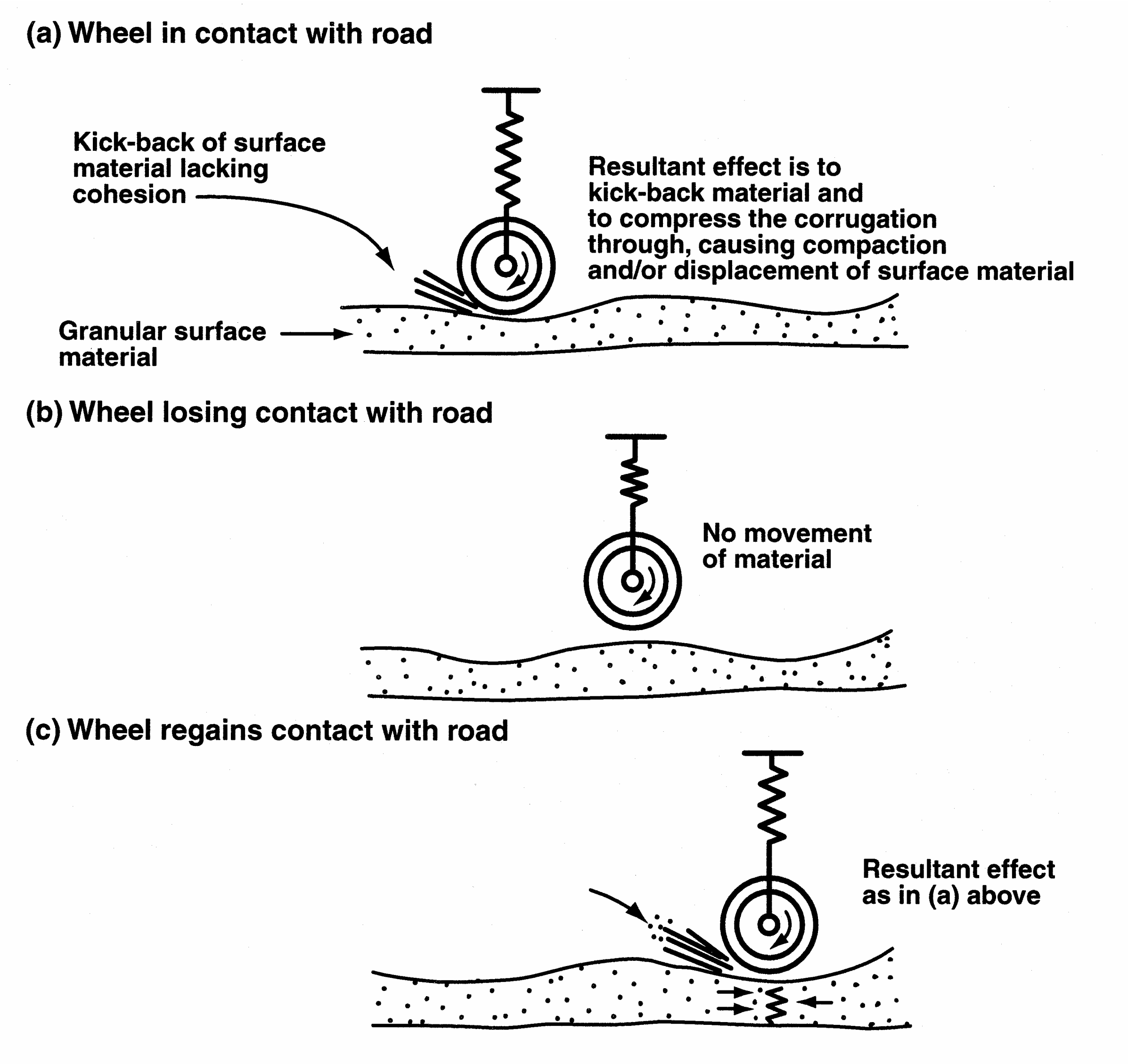


Figure 1-5: The Forced Oscillation Theory for the Formation of Corrugation

A number of the typical defects can be mitigated by an adequate selection of the materials for gravel wearing courses, which should satisfy the following requirements that are often somewhat conflicting:

(a) They should have sufficient cohesion to prevent ravelling and corrugating (especially in dry conditions)

(b) The amount of fines (particularly plastic fines) should be limited to avoid a slippery surface under wet conditions

These aspects are dealt with in the Specifications and are naturally influenced by the availability of materials. In design, the thickness requirements for the gravel wearing course will essentially derive from the combined need to protect the subgrade and to periodically replace the lost materials.

1.4 Design Process

The organization of this Manual is as presented in Figure 1-7 at the end of this chapter. The main steps involved in designing a new road pavement are as presented below and given in Figure 1-6):

- Surveying possible route (usually part of the feasibility study, see Route Corridor Selection Chapter in *Geometric Design Manual-2002*);
- estimating the traffic in terms of the cumulative number of equivalent standard axles that will use the road over the selected design life (cf. Chapter 2);
- characterizing the strength of the subgrade soil over which the road is to be built (cf. Chapter 3);
- selecting an adequate pavement structure, i.e. pavement materials and layer thicknesses providing satisfactory service over the design life, utilizing the catalog of pavement structures presented in Chapter 10 for flexible pavement, and the design process presented in Chapter 11 for gravel pavements. The structures given in this manual are based primarily on results of full-scale experiments and studies of the performance of as-built existing road networks.

Intermediate chapters of the manual re: Chapters 4 and 5, which give guidance and background information related to the soils, shoulder design, drainage, and cross section assumptions underlying the design of the structures presented; Chapters 6 to 9, similarly, provide guidance regarding the materials of the various pavement layers.

1.5 Variability and Reliability

1.5.1 Traffic

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

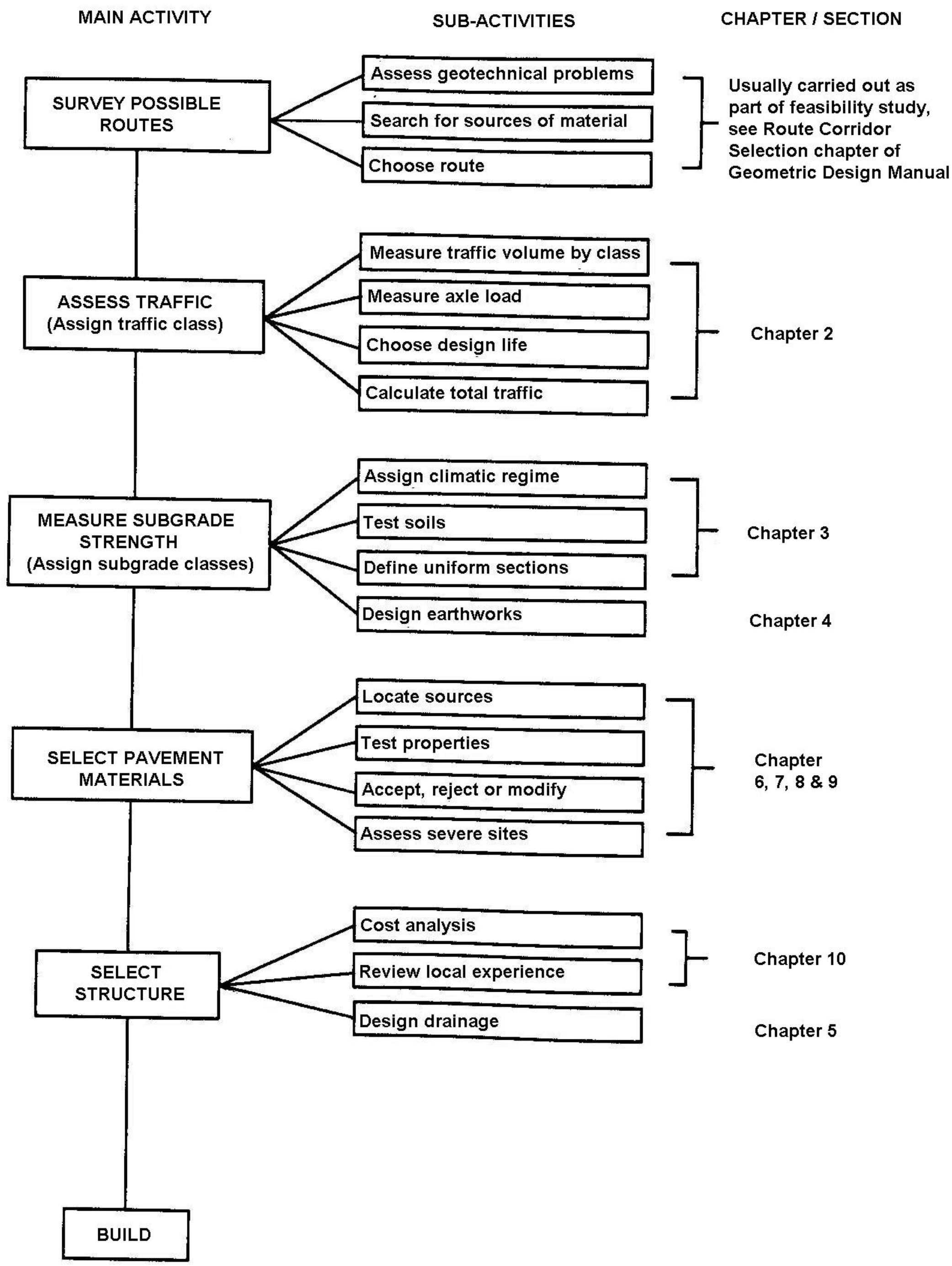


Figure 1-6: Pavement Design Process

1.5.2 Climate

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

The climate influences the subgrade moisture content and strength (cf. Chapter 3) and requires precautions to ensure adequate drainage (Chapter 5). The rainfall also influences the selection of adequate pavement materials, such as the allowable limits of materials properties (cf. Chapter 6), and is a potential incentive to use stabilized materials (cf. Chapter 7). The temperature influences the selection and design of bituminous surfacings (Chapters 8 and 9).

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

1.5.3 Materials

The properties of the materials are variable, and construction control is enforced with varying success. As mentioned elsewhere, expectations from the users play a role in defining acceptable levels of riding quality. By the same token, even if only a small percentage of the surface of a road shows distress, the road may be considered unacceptable. As a result, the weakest parts of the road are very important in design and identifying these parts and the variability of the pavement components similarly important. This argues strongly against minimizing the extent of preliminary investigations to determine this variability.

Changes in the subgrade strength are usually considered first, and other factors are assumed to be controlled by enforcing specifications (i.e. minimum acceptable values for key characteristics of the pavement materials). Even so, a considerable variation in performance between a priori identical pavements is often observed, which cannot be fully explained. An optimum design therefore remains partly dependent on knowledge of the performance of in-service roads and quantification of the variability of the observed performance itself (elements of pavement management systems). As a result, designs integrating local experience usually perform better.

The pavement structures given in this manual should be regarded essentially with the layers thicknesses and materials strength requirements as being minimum values. From a practical viewpoint, however, they may be interpreted as lower ten percentile values, i.e. with 90% of all test results exceeding the values quoted. Random variations in thickness and strength should be such that minor deficiencies in thickness or strength do not occur concomitantly, or very rarely so. Good construction practices to ensure this randomness and also to minimize variations themselves cannot be over emphasized.

The design process of flexible pavements must include an evaluation of the available materials in order to allow a selection among the viable alternatives. Similarly, for gravel roads, the availability of materials suitable as gravel wearing course needs to be verified.

The design of flexible pavements in this manual offers alternatives given in a catalog of pavement structures presented in Chapter 10 and discussed in Chapters 8 and 9. Gravel wearing courses are covered under Chapters 6 and 11.

1.5.4 Main Characteristics of Major Material Types: Granular Materials

Granular materials include selected fill layer; gravel subbase, roadbase or wearing course; and crushed stone subbase or roadbase. These materials exhibit stress dependent behavior, and under repeated stresses, deformation can occur through shear and/or densification.

The selected fill, compacted at 95% MDD (AASHTO T180) exhibits a minimum soaked CBR of 10%. Its minimum characteristics are specified by a minimum grading modulus (0.75) and maximum plasticity index (20%) (see Appendix A).
The gravel subbase and roadbase materials have minimum soaked CBRs of 30% and 80% respectively, when compacted to 95% and 98% MDD respectively. They are subject to requirements regarding grading modulus and plasticity index. In addition, the roadbase materials must satisfy requirements regarding particle shape, Ten Percent Fines value, Los Angeles Abrasion value and grading (see Appendix A).

The gravel wearing course materials should have sufficient cohesion and, simultaneously, a limited amount of plastic fines. The materials must satisfy requirements regarding minimum soaked CBR (20% at 95% MDD), Los Angeles abrasion value, particle shape, and grading.

Crushed stone materials are produced entirely by the crushing of rock or boulders and subject to strict grading requirements. The CBR need not be explicitly specified and only the compaction is controlled (95% and 98% MDD for subbase and roadbase, respectively). Other requirements include: Los Angeles Abrasion Value; flakiness index, percentage of crushed particles, plasticity index, and for roadbase materials, aggregate crushing value and sodium sulfate soundness value (see Appendix A).

1.5.5 Main Characteristics of Major Material Types: Bituminous Materials

Bituminous materials include bituminous concrete pavement layers; bituminous stabilization for roadbase; and dense bitumen macadam for roadbase. Bituminous materials are viscoelastic and under repeated stresses may either weaken or deform or both.

Bituminous concrete, i.e. asphalt concrete, for wearing and binder courses of surfacings, is a dense, continuously graded mix relying on the aggregate interlock and the bitumen properties for its strength. The mix is designed for durability and fatigue behavior.

Bituminous stabilization can be used for roadbase materials based largely on local experience and subject to construction of trial sections.

Dense bitumen macadams for use as roadbase are continuously graded mixes with an aggregate structure less dense than asphalt concrete.

1.5.6 AIN CHARACTERISTICS OF MAJOR MATERIAL TYPES: CEMENT OR LIME STABILIZED MATERIALS

Cement or lime stabilized materials include cement or lime stabilized subbase or roadbase

Materials stabilized with cement or lime, for use as subbase or roadbase, are elastic and possess tensile strength. They usually crack under repeated flexure, and also because of shrinkage and drying. Advantages of stabilized materials include the fact that they retain a substantial proportion of their strength when saturated, that the surface deflections of the pavement are reduced, and that the underlying materials cannot contaminate the stabilized layer. On the other hand, the tendency of these materials to crack may induce reflection cracks in the surfacing.

The selection of an appropriate stabilizer is made on the basis of the plasticity and grading of the materials to be treated. The stabilized materials exhibit increased strength and the required percentage of stabilizer is determined in the laboratory, with a view to achieve CBRs on the order of 40 and 80 - 100 for subbase and roadbase, respectively.

1.5.7 MAIN CHARACTERISTICS OF MAJOR MATERIAL TYPES: SURFACE TREATMENTS AND SEALS

Double seal bituminous surface treatments are most commonly used in connection with the catalog of pavement thickness. They consist of the application of two successive seals, each including the application of a bituminous binder followed by the application of chippings entirely produced by crushing stone, boulder or gravels. The application of chippings corresponds to selected combinations, of chipping sizes with specified grading requirements. Also specified are the flakiness index and the soundness of the chippings.

Guidance relative to the selection of combination(s) of chipping sizes is to be found in the ERA *Site Investigation Manual-2002*.

Single seals for new pavements may also be used over bituminous stabilized roadbases, for structures expected to carry medium levels of traffic. The single seals may be used in combination with a slurry (Cape Seal).

1.6 Economic Considerations

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

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

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

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

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

Illustrative examples of slab thickness design are given in this manual for concrete pavements. The implementation of such designs, if contemplated, should be further justified by comparative life-cycle cost analyses.

1.7 Basis for the Design Catalog

The pavement designs presented in this manual are based primarily on results of full-scale experiments and studies of the performance of as-built existing road networks.

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

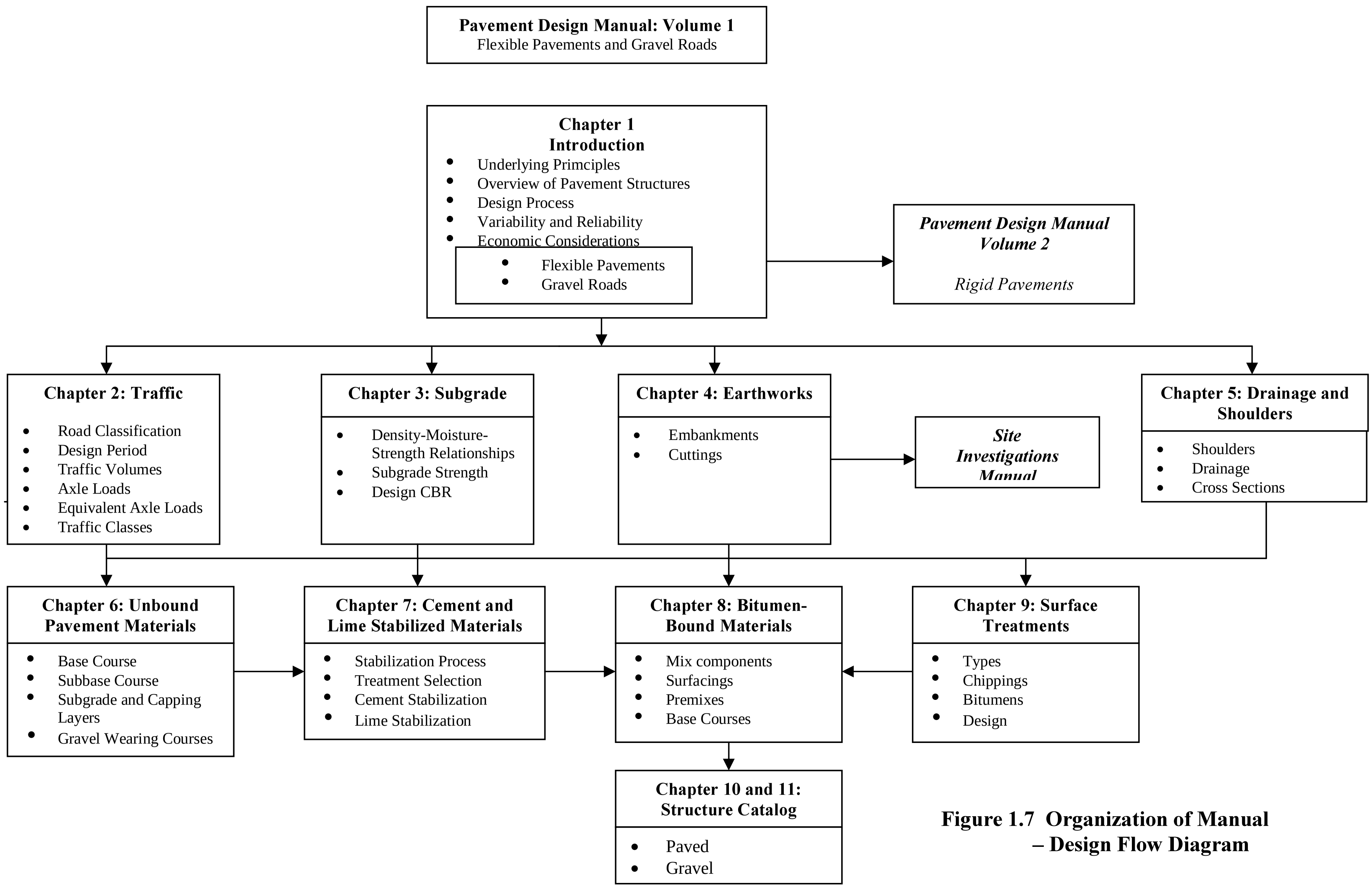


Figure 1.7 Organization of Manual – Design Flow Diagram

2. TRAFFIC

2.1 General

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

The mechanism of deterioration of **gravel roads** differs from that of paved roads and is directly related to the number of vehicles using the road rather than the number of equivalent standard axles. The traffic volume is therefore used in the design of unpaved roads, as opposed to the paved roads which require the conversion of traffic volumes into the appropriate cumulative number of equivalent standard axles.

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

2.2 Design Period

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

- Functional importance of the road
- Traffic volume
- Location and terrain of the project
- Financial constraints
- Difficulty in forecasting traffic

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

Problems in traffic forecasting may also influence the design. When accurate traffic estimates cannot be made, it may be advisable to reduce the design period to avoid costly overdesign.

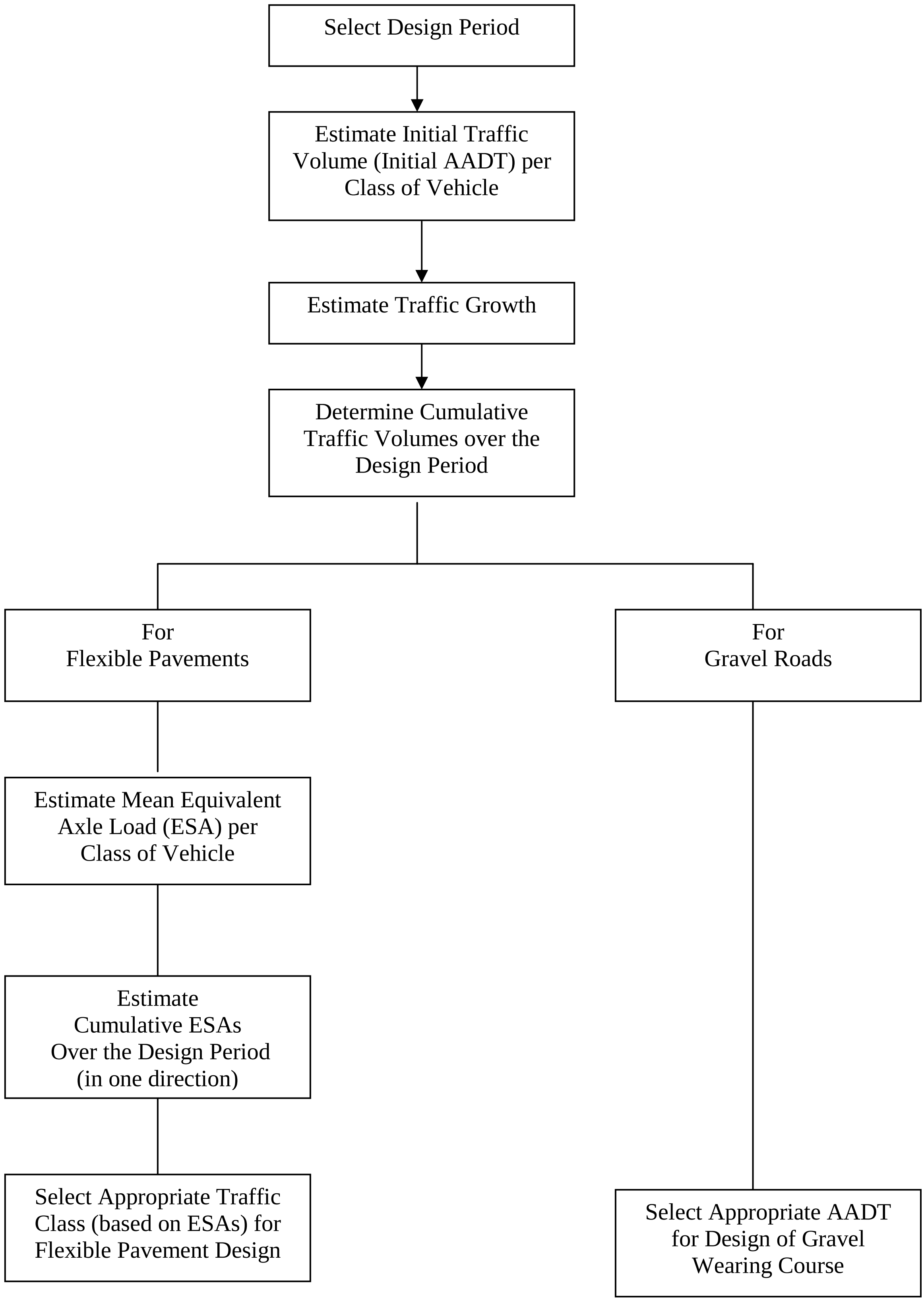


FIGURE 2-1 TRAFFIC EVALUATION

Bearing in mind the above considerations, it is important that the designer consults ERA at the outset of the project to ascertain the design period. Table 2-1 shows the general guidelines:

Table 2-1: Design Period

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

2.3 Traffic Volumes

2.3.1 Vehicle Classification

Vehicle classification is an essential aspect of traffic volume evaluation (as well as evaluation of equivalent axle loads). The types of vehicles are defined according to the breakdown adopted by ERA for traffic counts: cars; pick-ups and 4-wheel drive vehicles such as Land Rovers and Land Cruisers; small buses; medium and large size buses; small trucks; medium trucks; heavy trucks; and trucks and trailers. This breakdown is further simplified, for reporting purposes, and expressed in the five classes of vehicles (with vehicle codes 1 to 5) listed in Table 2-2.

Table 2-2: Vehicle Classification

Vehicle Code	Type of Vehicle	Description
1	Small car	Passenger cars, minibuses (up to 24-passenger seats), taxis, pick-ups, and Land Cruisers, Land Rovers, etc.
2	Bus	Medium and large size buses above 24 passenger seats
3	Medium Truck	Small and medium sized trucks including tankers up to 7 tons load
4	Heavy Truck	Trucks above 7 tons load
5	Articulated Truck	Trucks with trailer or semi-trailer and Tanker Trailers

It is most often in terms of volumes (e.g. AADT) in each of these 5 classes that the traffic data will initially be available to the designer. As mentioned before, small cars do not contribute significantly to the structural damage, particularly for paved roads. Even though the small cars count is included in any regular traffic count survey, their number does not influence the pavement design of paved roads. It is also worth noting that the “heavy” vehicles used in the development of the pavement structures essentially correspond, for all practical design purposes, to vehicle codes 2 through 5.

2.3.2 *INITIAL TRAFFIC VOLUMES*

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

Based on the review of various traffic studies conducted in Ethiopia, over the past 15 years, it can be concluded that the reported traffic volumes are very erratic. The traffic volumes do not indicate any specific trend. This makes it all the more difficult to predict volumes. Some practical constraints in enforcing accurate traffic surveys were also reported.

Because of the above constraints, a very thorough and conservative traffic count survey shall be taken up, in particular for all major and heavy traffic roads.

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

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

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

i. The counts are for seven consecutive days.
ii. The counts on some of the days are for a full 24 hours, with preferably at least one 24-hour count on a weekday and one during a weekend. On the other days 16-hour counts should be sufficient. These should be extrapolated to 24-hour values in the same proportion as the 16-hour/24-hour split on those days when full 24-hour counts have been undertaken.

iii. Counts are avoided at times when travel activity is abnormal for short periods due to the payment of wages and salaries, public holidays, etc. If abnormal traffic flows persist for extended periods, for example during harvest times, additional counts need to be made to ensure this traffic is properly included.

iv. If possible, the seven-day counts should be repeated several times throughout the year. Countrywide traffic data should preferably be collected on a systematic basis to enable seasonal trends in traffic volumes to be quantified. Presently, classified traffic counts are normally obtained by counting manually.

2.3.3 TRAFFIC FORECAST

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

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

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

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

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

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

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

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

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

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

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

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

The recommended approach to forecasting generated traffic is to use demand relationships.

Some studies carried out in similar countries give an average for the price elasticity of demand for transport of about -1.0. This means that a one per cent decrease in transport costs leads to a one per cent increase in traffic.

Note: At this stage, the designer has the required elements to determine the initial and forecast AADT. For paved roads, it is still necessary to consider the axle loads in order to determine the cumulative equivalent standard axle loads (ESA) over the design period (see Section 2.4 below) in order to select an appropriate traffic class (Section 2.4). For unpaved roads, as indicated earlier, only AADTs are required: the design AADT can be determined in a similar fashion as for paved roads using only Steps 1 to 3 of 5 of Section 2.4 and select the corresponding traffic class in Section 2.5.

2.3.4 Determination of Cumulative Traffic Volumes

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

1. Determine the initial traffic volume ($AADT_0$) using the results of the traffic survey and any other recent traffic count information that is available. For paved roads, detail the AADT in terms of car, bus, truck, and truck-trailer.

2. Estimate the annual growth rate “i” expressed as a decimal fraction, and the anticipated number of years “x” between the traffic survey and the opening of the road.

3. Determine $AADT_1$ the traffic volume in both directions on the year of the road opening by:

$$AADT_1 = AADT_0 \ (1+i)^x$$

For paved roads, also determine the corresponding daily one-directional traffic volume for each type of vehicle.

4. The cumulative number of vehicles, T over the chosen design period N (in years) is obtained by:

$$T = 365\ AADT_1\ [\ (1+i)^N - 1]\ /\ (\ i\ )$$

For paved roads, conduct a similar calculation to determine the cumulative volume in each direction for each type of vehicle.

2.4 Axle Loads

2.4.1 AXLE EQUIVALENCY

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

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

2.4.2 AXLE LOAD SURVEYS

Axle load surveys must be carried out to determine the axle load distribution of a sample of the heavy vehicles (vehicles with codes of 2 to 5) using the road. Data collected from these surveys are used to calculate the mean number of ESA for a typical vehicle in each class. These values are then used in conjunction with traffic forecasts to determine the predicted cumulative equivalent standard axles that the road will carry over its design life.

Most of the countries have regulations on the size and weight of vehicles to ensure road safety and to contain the weight of vehicles within the carrying capacity of the road pavements and bridges. However, in developing countries like Ethiopia, enforcement has usually proved to be quite impracticable. Vehicles are grossly overloaded. Examples were reported where axle loads are as much as 60 per cent higher than those permitted in the regulations. In such cases, a pavement design which assumes that the vehicles would be conforming to the country's regulations on vehicle weight and axle loading is bound to fail.

Hence, it is emphasized here that the designer should consider the factors:

1. Overloaded vehicles using the road
2. Ability to undertake effective road maintenance in his pavement design analysis on case by case basis.

The types of construction must be robust, capable of carrying the heavy loads, as far as possible, be capable withstanding some neglect of routine and periodic maintenance.

No regular axle load surveys are conducted in Ethiopia at present. Each individual project depends on its own axle load survey data. As mentioned earlier, since these surveys are for a limited time period, they may not give a representative data. Hence it is recommended that, a very thorough and conservative axle load survey over extended periods be carried out to determine the axle loads as accurately as possible. The accuracy of these surveys will have influence on the determination of traffic class.

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

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

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

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

The following method of analysis is recommended:

a. Determine the equivalency factors for each of the wheel loads measured during the axle load survey, using Table 2-3 or the accompanying equation, in order to obtain the equivalency factors for vehicle axles. The factors for the axles are totaled to give the equivalency factor for each of the vehicles. For vehicles with multiple axles i.e. tandems, triples etc., each axle in the multiple group is considered separately.

b. Determine the mean equivalency factor for each class of heavy vehicle (i.e. bus, truck and truck-trailer) travelling in each direction. It is customary to assume that the axle load distribution of the heavy vehicles will remain unchanged for the design period of the pavement.

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

2.4.3 Cumulative Equivalent Standard Axles over the Design Period

Finally, the cumulative ESAs over the design period (N) are calculated as the products of the cumulative one-directional traffic volume (T) for each class of vehicle by the mean equivalency factor for that class and added together for each direction. The higher of the two directional values should be used for design.

The relationship between a vehicle's EF and its axle loading is normally considered in terms of the axle mass measured in kilograms. The relationship takes the form:

$$\text{Equivalency factor} = \left(\frac{\text{Axle}_i}{8160}\right)^n$$

where

$axle_i$ = mass of axle *i*

n = a power factor that varies depending on the pavement construction type and subgrade but which can be assumed to have a value of 4.5

and the standard axle load is taken as 8 160kg with the summation taken over the number of axles on the vehicle in question

A list of axle load equivalency factors in given in Table 2-3:

Table 2-3: Equivalency Factors for Different Axle Loads (Flexible Pavements)

Wheel load (single & dual) (10^3 kg)	Axle load (10^3kg)	Equivalency Factor (EF)
1.5	3.0	0.01
2.0	4.0	0.04
2.5	5.0	0.11
3.0	6.0	0.25
3.5	7.0	0.50
4.0	8.0	0.91
4.5	9.0	1.55
5.0	10.0	2.50
5.5	11.0	3.93
6.0	12.0	5.67
6.5	13.0	8.13
7.0	14.0	11.3
7.5	15.0	15.5
8.0	16.0	20.7
8.5	17.0	27.2
9.0	18.0	35.2
9.5	19.0	44.9
10.0	20.0	56.5

Notes: (1) The equivalency factors given in Table 2-3 are to be used solely in the context of this volume for flexible pavement design. Refer to Volume 2 for specific factors for rigid pavements.

(2) The equation used has been widely used for years, but was not developed under a range of loads and climatic and soils conditions representative of those prevailing in Ethiopia. Caution must therefore be exercised in assessing the results of its use and sensitivity analyses are recommended in final design.

When the pavement design is for carriageways with more than one traffic lane in each direction, a reduction may be considered in the cumulative ESA to take into account for the design. The ranges given in Table 2-4 are suggested for the percentage of design ESAs to consider in the design lane:

Table 2-4: Percentage ESAs per Lane for Multiple Lanes

Number of lanes in each direction	Percent of ESAs in design lane
1	100
2	80 – 100
3	60 – 80

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

2.5 Traffic Classes for Flexible Pavement Design

Accurate estimates of cumulative traffic are very difficult to achieve due to errors in the surveys and uncertainties with regard to traffic growth, axle loads and axle equivalencies.

To a reasonable extent, however, pavement thickness design is not very sensitive to cumulative axle loads and the method recommended in this manual provides fixed structures of paved roads for ranges of traffic as shown in Table 2-5. As long as the estimate of cumulative equivalent standard axles is close to the center of one of the ranges, any errors are unlikely to affect the choice of pavement design.

However, if estimates of cumulative traffic are close to the boundaries of the traffic ranges, then the basic traffic data and forecasts should be re-evaluated and sensitivity analyses carried out to ensure that the choice of traffic class is appropriate. As mentioned earlier, depending on the degree of accuracy achieved, a higher traffic class may be appropriate for some cases.

Table 2-5: Traffic Classes for Flexible Pavement Design

Traffic classes	Range (10^6 ESAs)
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

2.6 Accuracy- Traffic Classes

All survey data are subject to errors. Traffic data, in particular, can be very inaccurate and predictions about traffic growth are also prone to large errors. Accurate calculations of cumulative traffic are therefore very difficult to make. To minimize these errors there is no substitute for carrying out specific traffic surveys for each project for the durations suggested in Section 2.3. Additional errors are introduced in the calculation of cumulative standard axles because any small errors in measuring axle loads are amplified by the fourth power law relationship between the two.

Fortunately, pavement thickness design is not very sensitive to cumulative axle loads and the method recommended in this manual provides fixed structures of paved roads for ranges of traffic as shown in Table 2-5. As long as the estimate of cumulative equivalent standard axles is close to the center of one of the ranges, any errors are unlikely to affect the choice of pavement design. However, if estimates of cumulative traffic are close to the boundaries of the traffic ranges, then the basic traffic data and forecasts should be re-evaluated and sensitivity analyses carried out to ensure that the choice of traffic class is appropriate. As mentioned in section 2.3 and 2.4, depending on the degree of accuracy achieved, higher traffic class may be appropriate for some cases.

It is recommended that for the highest traffic class for unpaved roads (T4), a verification of the cumulative number of equivalent axle loads be carried out as for paved roads, in order to determine in which traffic class of paved road a particular road project would fall. Consideration should be given to paving if the evaluation indicates a traffic class of paved road higher than T1. No strict higher limit of traffic is given for the traffic class T4 for unpaved roads, but the recommendations given herein are generally considered to be for traffic levels below an AADT of 500 vehicles per day in both directions.

2.7 Design Example

Initial traffic volumes in terms of AADTs have been established for 2002 for a section of a trunk road under study, as follows:

Vehicle classification	2002 AADT
Car	250
Bus	40
Truck	130
Truck-trailer	180

The anticipated traffic growth is a constant 5%, and the opening of the road is scheduled for 2005. In addition, an axle load survey has been conducted, giving representative axle loads for the various classes of heavy vehicles, such as given below for truck-trailers (it is assumed that the loads are equally representative for each direction of traffic):

Vehicle No	Axle loads (Kg) Axle 1	Axle 2	Axle 3	Axle 4
1	6780	14150	8290	8370
2	6260	12920	8090	9940
3	6350	13000	8490	9340
4	5480	12480	7940	9470
5	6450	8880	6290	10160
6	5550	12240	8550	10150
7	5500	11820	7640	9420
8	4570	13930	2720	2410
9	4190	15300	3110	2450
10	4940	15060	2880	2800

The projected AADTs in 2005 can be calculated as (AADTs in 2002) x $(1.05)^3$, and the corresponding one-directional volumes for each class of vehicle in 2005 are:

Vehicle classification	One-directional traffic volume in 2005
Car	145
Bus	23
Truck	75
Truck-trailer	104

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

Vehicle classification	Cumulative no. of vehicles in one direction over 20 years
Car	$365x145[(1.05)^{20}-1]/0.05=1750016$
Bus	$365x23[(1.05)^{20}-1]/0.05=277589$
Truck	$365x75[(1.05)^{20}-1]/0.05=905180$
Truck-trailer	$365x104[(1.05)^{20}-1]/0.05=1255184$

Equivalency factors for the sample of truck-trailers, and a mean equivalency factor for that class of heavy vehicles, can be calculated as outlined below:

Vehicle No	Axle 1 Load	Axle 1 Factor	Axle 2 Load	Axle 2 Factor	Axle 3 Load	Axle 3 Factor	Axle 4 Load	Axle 4 Factor	Total Factor
1	6780	0.43	14150	11.91	8290	1.07	8370	1.12	14.54
2	6260	0.30	12920	7.91	8090	0.96	9940	2.43	11.60
3	6350	0.32	13000	8.13	8490	1.20	9340	1.84	11.49
4	5480	0.17	12480	6.77	7940	0.88	9470	1.95	9.77
5	6450	0.35	8880	1.46	6290	0.31	10160	2.68	4.80
6	5550	0.18	12240	6.20	8550	1.23	10150	2.67	10.28
7	5500	0.17	11820	5.30	7640	0.74	9420	1.91	8.12
8	4570	0.07	13930	11.10	2720	0.01	2410	0.00	11.18
9	4190	0.05	15300	16.92	3110	0.01	2450	0.00	16.99
10	4940	0.10	15060	15.76	2880	0.01	2800	0.01	15.88
			Mean equivalency factor for truck-trailers =						11.47

For the sake of this example, it will be assumed that similar calculations have been performed, giving mean equivalency factors for buses and trucks of 0.14 and 6.67 respectively.

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

Vehicle classification	Cum. no. of vehicles	Equivalency factor	10^6 ESAs
Car	1750016	0.00	0.0
Bus	277589	0.14	0.0
Truck	905180	6.67	6.0
Truck-trailer	1255184	11.47	14.4
		Total ESAs =	20.4

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

2.8 Estimating Axle Loads for Gravel Roads

It is unlikely to be cost effective to carry out axle load surveys on gravel and low standard in Ethiopia. In such circumstances, the default values given in Table 2.6 should be used:

Table 2-6: Default Values for Axle Load Factors for Gravel and Low Standard Roads

Axles per Heavy Vehicle	2.3
ESAs per Heavy Axle	0.20
ESAs per Heavy Vehicle	0.46

The implications of applying the default values for different design periods are shown in Table 2.7.

Default ESA values should be modified for specific local circumstances. For example, where a factory or mine exists near the project road, use values higher than the defaults, or an axle load survey can be done to provide a better estimate of ESAs per vehicle. A simpler option is to analyze the types of vehicles expected to use the road. General Guidance on likely equivalency factors for different vehicle types is given in Table 2.8.

Use the following procedure to determine the cumulative ESAs over the design life:

1. Determine the daily traffic flow of heavy vehicles using results from the traffic counts or other recent traffic survey data.
2. Determine the average daily one-directional traffic flow for heavy vehicles.
3. Using considerations of flows and growth rates for normal, diverted and generated traffic, forecast the one-directional traffic flow of heavy vehicles which will travel over the road during the design life.

4. Determine the mean EF for heavy vehicles in each *direction* of travel, and take the *higher* of these values.
5. The products of the cumulative one-directional traffic flows for heavy vehicles over the design life of the road and the mean EF, should then be calculated. This gives the cumulative ESAs for the heavier laden direction.

Note the following points in the calculation:

- The traffic used is that 1) for heavy vehicles only, and 2) traveling on one direction only (the most heavily laden direction)
- If axle load data are available from surveys, it is important that the EF for each vehicle is determined, and the mean found by averaging these values. Determining the EF of the mean axle load will seriously underestimate the true value.

Table 2-7: Cumulative Default ESAs for Different Design Periods

First Year Traffic		Cumulative Values after 10 Years		Cumulative Values after 15 Years		Cumulative Values after 20 Years	
AADT	Heavy vehicles per lane	Heavy vehicles per lane	ESAs	Heavy vehicles per lane	ESAs	Heavy vehicles per lane	ESAs
18	3	13 803	6 350	23 023	10 591	34 237	15 749
20	4	15 337	7 055	25 582	11 767	38 042	17 499
30	5	22 786	10 482	38 007	17 483	56 519	25 999
40	7	30 674	14 110	51 163	23 535	76 083	34 998
50	9	39 438	18 141	65 781	30 259	97 821	44 998
60	11	46 011	21 165	76 745	35 302	114 125	52 497
80	14	61 348	28 220	102 326	47 070	152 166	69 996
100	18	76 685	35 275	127 908	58 837	190 208	87 495
120	21	92 022	42 330	153 489	70 605	228 249	104 995
150	26	113 932	52 409	190 034	87 416	282 594	129 993
180	32	138 033	63 495	230 234	105 907	342 374	157 492
200	35	155 379	70 050	258 515	117 675	380 415	174 991
300	55	230 055	105 825	383 723	176 512	570 623	262 486
350	61	268 398	123 463	447 676	205 931	665 726	306 234
400	70	306 740	141 100	511 630	235 350	760 830	349 982
500	88	383 425	176 376	639 538	294 187	951 038	437 477

Notes:
1. Heavy vehicles per lane is assumed to be 35% of the one lane flow (ie (AADT x 0.35)/2)
2. Traffic assumed to grow at 4 per cent

Table 2-8: Equivalency Factors for Different Heavy Vehicle Configurations

Vehicle Type	Average ESAs per Vehicle	Typical Range of Average ESAs per Vehicle
2-axle truck	0.70	0.30 – 1.10
2-axle bus	0.73	0.41 – 1.52
3-axle truck	1.70	0.80 – 2.60
4-axle truck	1.80	0.80 – 3.00
5-axle truck	2.20	1.00 – 3.00

Estimates of baseline traffic flows, traffic growth rates and axle loading are subject to errors of 20% or more. Additional errors are introduced into the calculation of Cumulative ESAs because small errors in axle loads are amplified by the 4.5-power relationship between the two. To minimize these errors, carry out specific traffic and axle load surveys. The design methods in this manual tend to be conservative, so the lower standard design should always be adopted at the margin.

3. SUBGRADE

3.1 General

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

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

In the following sections, it has been considered useful to first recall some basic relationships involved in the modifications imposed on the subgrade soil during and after the road construction, and how they affect the final subgrade strength (Section 3.2). Next, in Section 3.3, the various steps leading to the selection of a design CBR are detailed.

3.2 General Density-Moisture Content-Strength Relationships of the Subgrade

As indicated above, the strength of the subgrade is assessed in terms of CBR. The CBR depends on the nature of the soil, its density and its moisture content.

The nature of the soil is dictated by the route location and the selected longitudinal profile for the road, and does not change except for the influence of the borrow materials and the movement of materials between cut and fill during earthworks operations.

By contrast, the (dry) density of the subgrade soil will be modified from its original state at the time of the road construction, by compaction at subgrade level in cuts and by compaction of the excavated materials used in embankments. Similarly, the moisture content of the natural subgrade soil will be altered during construction, in order to approach the optimum (indicated by laboratory tests) which is conducive to a greater increase in density and in corresponding CBR strength. Upon completion of the construction operations, the natural soils will have been brought to a second state of moisture, density and strength. This second state is not the final state of the subgrade, however, and except in few particular cases (see Category 1 below), should not be used in design.

Following the construction, the compacted subgrade soil will approximately keep the same dry density, except for compaction under traffic and possible volume variations of certain sensitive soils. However, even if the pavement was constructed immediately after finishing the subgrade and if the pavement could be considered perfectly waterproof, the moisture content of the subgrade would nevertheless evolve due to local soil, groundwater or seasonal conditions. It is this third ultimate state of the subgrade that generally needs to be considered in design.

To illustrate the above discussion, Figures 3-1 and 3-2 (adapted from Ref. 7) give examples of relationships between density, moisture content and CBR. Two figures are given to emphasize that the relationships are specific to the nature of the subgrade soil. The figures indicate a "likely level of compaction achieved during construction", i.e. the second stage mentioned above. It is easy to imagine an initial state of the natural soil, with lower density and CBR. It is also easy to imagine an increase (for instance) in moisture content following the construction, with a corresponding decrease in strength.

3.3 Design Subgrade Strength

To determine the subgrade strength to use for the design of the road pavement, it is apparent from the above that it is necessary to ascertain the density-moisture content-strength relationship(s) specific to the subgrade soil(s) encountered along the road under study. It is also necessary to select the density which will be representative of the subgrade once compacted. Estimating the subgrade moisture content that will ultimately govern the design, i.e. the moisture content following the construction, is also required. It is recommended to determine the moisture content as a first step in the process, as this could influence the subsequent ones.

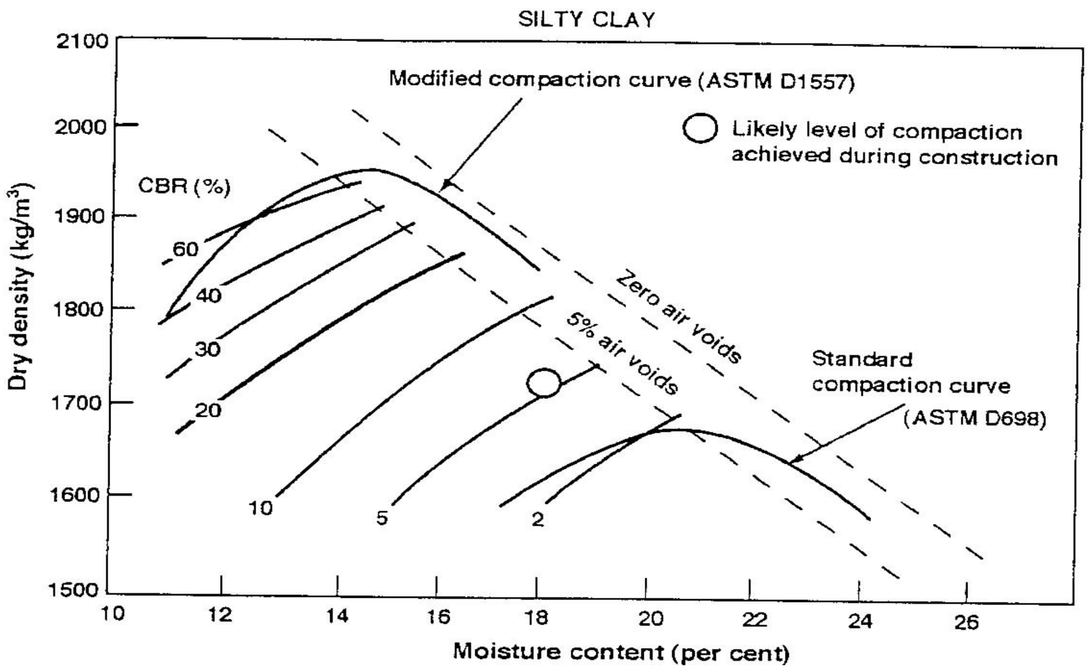


Figure 3-1: Dry Density, Moisture Content, Soil Strength Relationship for a Silty Clay

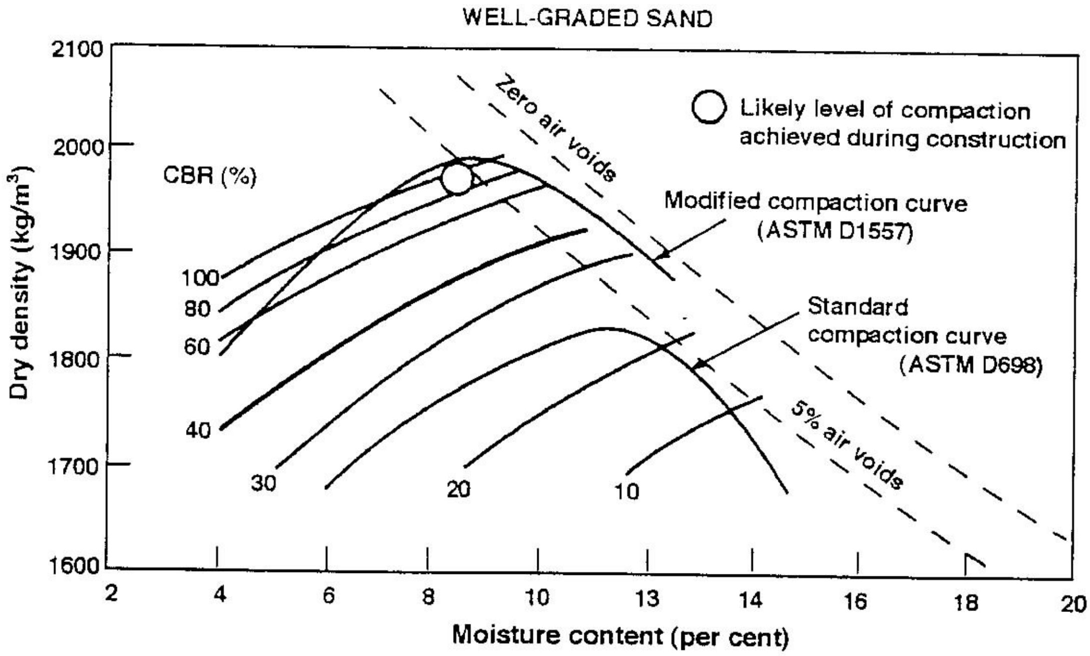


Figure 3-2: Dry Density, Moisture Content, Soil Strength Relationship for a Well-Graded Sand

3.3.1 ESTIMATED DESIGN MOISTURE CONTENT OF THE SUBGRADE

After the pavement is constructed, the moisture content of the subgrade will generally change.

In the dry southeast and northeast parts of Ethiopia, a decrease in the moisture content may be expected.

The moisture content, on the other hand, can increase elsewhere due to perched water tables during wet seasons. Also, in low-lying areas, the normal water table may be close to the finished subgrade level and influence the ultimate moisture content (whereas, with deep water tables and proper design and construction, it is less likely that the subgrade will get wetter after construction).

To approach the selection of the design moisture content, it is worth considering the classification into three conditions (cf. Ref. 1 and 7):

- Category 1: The water table is within 7 meters of the proposed road surface

In that case, the depth to the water table may govern the subgrade moisture.

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

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

As an alternate, if there is no suitable paved road in the vicinity, measurement of soil suction may be considered to determine the influence of the water table on the subgrade moisture content (as described in Appendix B). This method however requires that the apparatus and skilled personnel are available.

Another design approach alternative, if the water table level can be determined, is given further below (see Table 3-2). This approach may on occasions omit the determination of the moisture content and correlate directly the depth of water table and nature of the soil to an estimated subgrade strength class. This method is not as precise as a direct measurement, but can help in any event to verify that the results obtained are reliable.

Finally, in some cases such as in particularly low lying areas, or where it is determined or strongly suspected that the water table is close to the subgrade finished level, it is appropriate to consider that the moisture content will reach or approach saturation. The design strength may then be based on this assumption (design CBR based on testing soaked specimens).

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

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

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

- Category 3: Deep water table and arid climate

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

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

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

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

3.3.2 Representative Density

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

To specify densities during construction, it is recommended that the top 25 cm of all subgrades should be compacted to a relative density of at least 100% of the maximum dry density achieved by ASTM Test Method D 698 (light or standard compaction). Alternatively, at least 93% of the maximum dry density achieved by ASTM Test Method D 1557 may be specified. With modern compaction equipment, a relative density of 95% of the density obtained in the heavier compaction test should be achieved without difficulty, but tighter control of the moisture content will be necessary.

As a result, it is generally appropriate to base the determination of the design CBR on a density of 100% of the maximum dry density achieved by ASTM Test Method D 698 (light or standard compaction) or, alternately, on 93% of the maximum dry density achieved by ASTM Test Method D 1557 (heavy or modified compaction).

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

3.3.3 Specific Density-Moisture Content- Strength Relationships

The ERA *Site Investigation Manual-2002* recommends as a first step to conduct standard compaction tests (ASTM D 698) and to measure the CBR on samples molded at 100% MDD and OMC (standard compaction), to guide in the selection of homogeneous sections of a road project.

Following this selection, each typical soil is subjected to a more detailed testing involving three levels of compaction, and, at each level, two conditions of moisture.
The design CBR is then obtained by interpolation as illustrated below. This method enables an estimate to be made of the subgrade CBR at different densities and allows the effects of different levels of compaction control on the structural design to be evaluated.

3.3.4 Design CBR and Design Subgrade Strength Class

Figure 3-3 shows a detailed dry density/moisture content/CBR relationship (adapted from Ref. 1) for a sandy-clay soil that was obtained by compacting samples at several moisture contents to three levels of compaction. By interpolation, a design subgrade CBR of about 15 per cent is obtained if a relative density of 100 per cent of the maximum dry density obtained in the ASTM Test Method D 698 Test is specified and the subgrade moisture content was estimated to be 20 percent.

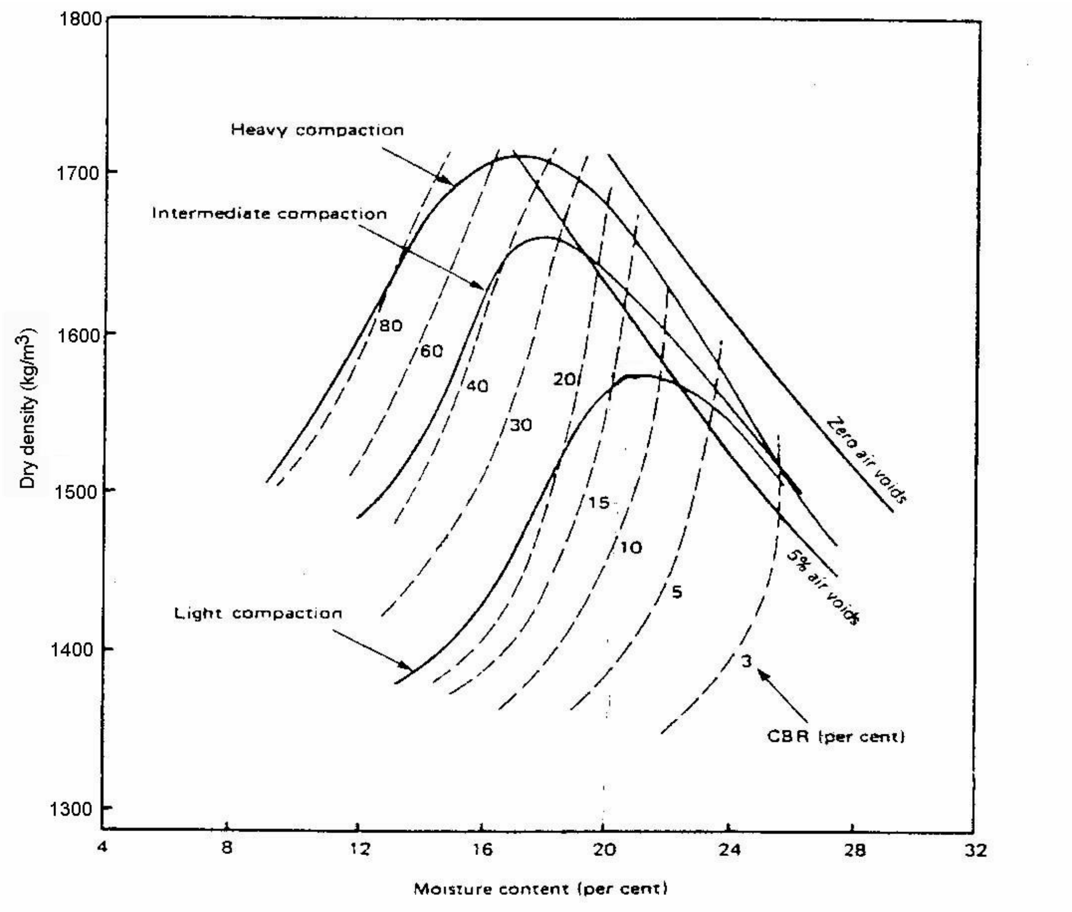


Figure 3-3: Dry Density, Moisture Content-CBR Relationships for Sandy-Clay Soil

The procedure outlined above and also detailed in the ERA *Site Investigation Manual-2002* is not as elaborate as to give complete interpolation curves as shown in Figure 3-3, but is nevertheless sufficient to conduct the necessary interpolations. This laboratory determination is the first (and generally preferred) option available to obtain a design CBR.

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

The structural catalog given in this manual requires that the subgrade strength for design be assigned to one of six strength classes reflecting the sensitivity of thickness design to subgrade strength. The classes are defined in Table 3-1. For subgrades with CBRs less than 2, special treatment is required (which is not covered in this manual).

Table 3-1: Subgrade Strength Classes

Class	Range (CBR %)
S1	2
S2	3 – 4
S3	5 – 7
S4	8 – 14
S5	15 – 29
S6	30+

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

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

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

* The highest seasonal level attained by the water table should be used.

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

Note: Since the strength classes given in Table 3-2 are based on estimated minimum CBR values, wherever possible the CBR should be measured by laboratory testing at the appropriate moisture content.

1. Table 3-2 is not applicable for silt, micaceous, organic or tropically weathered clays. Laboratory CBR tests should be undertaken for these soils.

2. A more detailed table relating soil type, minimum design CBR and depth of water table may be found in Ref. 7.

3.3.5 Delineation of Subgrade Areas

A road section for which a pavement design is undertaken should be subdivided into subgrade areas where the subgrade CBR can be reasonably expected to be uniform, i.e. without significant variations. Significant variations in this respect mean variations that would yield different subgrade classes as defined herein further below. However, it is not practical to create a delineation between subgrade areas that would be too precise, and indeed this could be the source of confusion during construction. The soils investigations should delineate subgrade design units on the basis of geology, pedology, drainage conditions and topography, and consider soil categories which have fairly consistent geotechnical characteristics (e.g. grading, plasticity, CBR). Usually, the number of soil categories and the number of uniform subgrade areas will not exceed 4 or 5 for a given road project. Generally, it is advisable to avoid short design sections along the alignment. Where the subgrade CBR values are very variable, the design should consider the respective benefits and costs of short sections and of a conservative approach based on the worst conditions over longer sections.

It is important to differentiate between localized poor (or good) soils and general subgrade areas. Normally, localized poor soils will be removed and replaced with suitable materials.

Lateritic gravels can generally be assigned a subgrade classification S5. It must be emphasized that too many variables influence the subgrade strength for the above to be anything more than a general indication, and detailed investigations, as outlined in the ERA *Site Investigation Manual-2002* are required for final design.

Other useful correlations for assessing qualitatively the subgrade strength include: a correlation between the nature of the soils (as given in the Unified Soil Classification System, USCS, described in ASTM Method D2487) and typical design CBR values; and the use of the AASHTO classification. By nature, these classifications cover all soils encountered in Ethiopia. The correlation between the nature of the soil and typical design CBR values is given in Table 3-3.

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

Table 3-3: Typical Design CBR Values (adapted from Ref. 10)

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

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

4. EARTHWORKS

4.1 Introduction

In this chapter, guidelines are given which pertain both to geotechnical design of the roadway and more specifically to pavement design. The geotechnical considerations are mentioned only to the extent that they participate in a comprehensive design; however, the guidelines given herein are not intended to replace a comprehensive geotechnical design. Geotechnical design per se is beyond the scope of this manual, which deals with pavements.

Considerations relative to materials and compaction, for instance, are more directly related to pavement design and performance than slope stability.

It is also to note that some considerations relative to earthworks belong to a manual dealing with soils rather than one dealing with pavements. For this reason, this chapter should be read in conjunction with ERA's *Site Investigation Manual-2002*.

4.2 Embankments

4.2.1 GENERAL

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

- Foundations conditions, with their associated potential problems of settlements and stability
- Embankment materials and related topics regarding specified methods of placing and compaction. Protection of the completed embankment slopes is equally important.

Potential problems in these areas should be identified during the reconnaissance, as described in ERA's *Site Investigation Manual-2002*.

4.2.2 EMBANKMENT FOUNDATIONS

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

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

4.2.3 EXPANSIVE SOILS

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

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

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

Mitigation measures include the excavation and replacement of expansive soils, although his may be economically nonviable. Less expensive compromises include the provision of constant moisture contents over the full width of the carriageway, possibly through sealing of the shoulders, the replacement of the upper layer of the expansive soil, and provision of a minimum cover.

Side drains should be avoided in areas of expansive soils. Where this is not possible, they should be placed a minimum distance away from the toe of the side slope, as indicated in the *ERA Geometric Design Manual - 2002*. The side slope should also be increase to a minimum of 1:6, as per the *ERA Geometric Design Manual - 2002*.

During construction, the roadbed of expansive soil should be kept moist and covering with earthworks prior to any drying. Attempts to process and compact the soil beyond normal density requirements is not required. Fill material over the expansive soils shall be impermeable soils with a plasticity index of greater than 15%.

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

4.2.4 Embankment Settlements

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

It is rather important to distinguish between two types of soft and compressible soils:

- Under-consolidated silt-clay mixtures
- Organic soils

The difference lies in their consolidation characteristics.

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

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

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

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

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

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

4.2.5 EMBANKMENT STABILITY

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

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

It is also useful to note that there exist a variety of simplified methods and design charts which will give a fair approximation of the safety factor under common conditions (e.g. Ref. 8).

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

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

- Cohesionless sands: 1: 3 if h <= 1 m
  1: 2 if h > 1 m
- Other materials: 1: 3 if h <= 1 m
  1: 2 if 1 m < h < 3 m
  1: 1.5 if 3 m < h < 10 m

where h is the height of the embankment.

In particularly wet areas of Ethiopia, it may be desirable however to use flatter slopes when the embankments are silty or clayey.

Steeper slopes in combination with reinforcement of the embankment material may become of value in certain urban sites.

4.2.6 Types of Embankment Materials

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

Most soils are suitable for embankment construction and the use of the majority of available materials should be encouraged.

Some soils are however generally unsuitable:

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

Exceptions may be made to the above, on a case by case basis. For instance, when alternatives are prohibitively expensive, black cotton soils may be used, provided methods to alleviate their associated problems are effected (cf. *Site Investigation Manual-2002*).

Rockfill may be used to form the base of the embankments in uniform layers not exceeding 1 meter in thickness (oversize materials to be reduced in size). Voids in the top layer (30 cm) of rock should be filled. Rock in embankments should not reach above an elevation 60 cm below the finished subgrade.

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

4.2.7 PLACING AND COMPACTION OF EMBANKMENT MATERIALS

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

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

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

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

As indicated in Section 3.3, it is recommended that the upper 25 cm of soil immediately beneath the subbase or capping layer, i.e. the top of the embankment fill or the natural

subgrade, be compacted to a minimum of 100 % of the maximum dry density obtained by ASTM D 698 (standard compaction). Alternatively, 93 % of the maximum dry density achieved by ASTM Test Method D 1557 (heavy compaction) may be specified. The same density should also be specified for fill behind abutments to bridges and for the backfill behind culverts. For the lower layers of an embankment, a compaction level of

90-93 per cent of the maximum dry density obtained by the heavy compaction is suitable, or a level of 95-100 per cent of the maximum density obtained by the light compaction. During construction, compaction trials are to be carried out to determine the best way to achieve the specified density with the equipment available. Also during construction, it is not always easy to obtain an accurate measure of field density on site. The standard traditional methods of measurement are tedious, not particularly reproducible, and it is difficult to carry out sufficient tests to define a reliable density distribution. This problem can be alleviated to a great extent by making use of nuclear density and moisture gauges, since such devices are quicker and the results are more reproducible than traditional methods. However, the instruments will usually need calibration for use with the materials in question if accurate absolute densities are required. It may also be advisable to measure the moisture contents using traditional methods.

4.2.8 Slope Protection

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

4.3 Cuttings

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

4.3.1 Slope Stability

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

Additional considerations regarding slope stability in cut sections are given in the *ERA Site Investigation Manual - 2002.*

4.3.2 SURVEYS

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

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

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

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

4.3.3 DESIGN AND CONSTRUCTION

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

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

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

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

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

5. DRAINAGE AND SHOULDERS

5.1 Drainage System

Provision must be made for protecting the road from surface water or ground water. If water is allowed to enter the structure of the road, the pavement will be weakened and it will be much more susceptible to damage by traffic. Water can enter the road as a result of rain penetrating the surface or as a result of the infiltration of ground water. The road surface must be constructed with a camber so that is sheds rainwater quickly and 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.

A good road (external) drainage system, properly maintained, is essential to the successful performance of a road and the pavement designs described in this manual are based on the assumption that the side drains (see Section 5.2) and culverts associated with the road are properly designed and function correctly.

Drainage within the pavement layers themselves (internal drainage) is a critical element of the pavement design because the strength of the subgrade used for design purposes depends on the moisture content during the most likely adverse conditions (see Chapter 3). It is impossible to guarantee that road surfaces will remain waterproof throughout their lives, hence it is important to ensure that water is able to drain away quickly from within the pavement layers (see Section 5.3).

5.2 External Drainage

Provision must be made for protecting the road from surface water or ground water. If water is allowed to enter the structure of the road, the pavement will be weakened and it will be much more susceptible to damage by traffic. Water can enter the road as a result of rain penetrating the surface or as a result of the infiltration of ground water. The road surface must be constructed with a camber so that it sheds rainwater quickly and 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.

A good road (external) drainage system, properly maintained, is essential to the successful performance of a road and the pavement designs described in this manual are based on the assumption that the side ditches and culverts associated with the road are properly designed and function correctly.

In order to exclude water from the road, the top of the shoulders should preferably be impermeable and a surface dressing or other seal may be applied to serve this purpose (see Chapter 9). Sealed shoulders also prevent the ingress of water at the edge of the pavement, which is an area particularly vulnerable to structural damage, particularly if the base course material lacks cohesion. A surfacing also helps protecting the shoulder against erosion.

The preferred solution consists of using a (usually single) surface dressing (see Chapter 9). This solution is particularly beneficial for road segments with high traffic. It is also one of two alternatives (together with the solution below using a prime coat) which is required for crushed stone shoulders.

Alternatively, a prime coat may provide some protection to the shoulders. A sanding may follow the priming of the surface of the shoulder. Variations to this solution (e.g. seals) are given in Chapter 9. Such a solution, or a surface dressing, is required for crushed stone shoulders and may be used for gravel shoulders.

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

Finally, if economics or local conditions warrant it, unsurfaced shoulders may be used, but will generally require maintenance and are not generally recommended. Unsurfaced shoulders must not be used if the materials are pervious (e.g. extended pervious base course). Unsurfaced shoulders may be provided with topsoiling and seeding. If gravel shoulders are left unsurfaced, the extra width given to the base course (over the road surface width) should nevertheless be primed and sealed (see Section 5.4). This edge seal should also extend over the shoulder.

Crossfall is needed on all roads in order to assist the shedding of water into the side ditches. A suitable value for paved roads is about 3% for the carriageway, with a slope of about 4% for the shoulders.

Note: A uniform cross slope of 4% is considered adequate for both wearing course and shoulders of unpaved (gravel) roads, where in any case materials are usually undistinguished.

5.3 Internal Drainage

Drainage within the pavement layers themselves (internal drainage) is a critical element of the pavement design. The strength of the subgrade used for design purposes depends on the moisture content during the most likely adverse conditions (cf. Chapter 3**)**. Since it is unlikely that road surfaces will remain waterproof throughout the design life of the pavement, it is important to ensure that water is able to drain away quickly from within the pavement.

Provided that the crossfalls indicated above are adhered to and the bituminous surfacing and the shoulders are properly maintained, rainwater falling on the road will run off adequately over the shoulders.

When permeable base course materials are used and in particular crushed stone bases (see Section 3.1 for permeability of base course material), particular attention must be given to the drainage of this layer. Under no circumstances should the “trench” type of cross-section be used in which the pavement layers are confined between continuous impervious shoulders.

When permeable bases are used, a distinction may be made depending on the nature of the subbase:

- If the subbase is relatively impervious, the preferred solution is to extend the base and subbase across the shoulders. An alternative solution consists of providing a drainage layer under the shoulder material (which may be gravel) at the level of the bottom of the base course. Although cheaper, this later solution is highly dependent on proper execution, and may not provide as much bearing capacity for the shoulder. Generally, the drainage layer should be omitted on the upper side of superelevated sections.

- When both the base and subbase are pervious, the preferred solution is again to extend both layers across the shoulders. An alternative consists of extending only the subbase course across the shoulder. This alternative may be effective if the subbase can be confidently considered pervious. Gravel may be used for the shoulders above the subbase.

When the base course can confidently be considered impervious, then the internal drainage is of lesser consequence. Impervious materials should still be used for the shoulder, and it is still preferable to provide them with surfacing. As mentioned previously, an effective seal should be provided between base and shoulder materials.

5.4 Shoulders

The width of the carriageway and the overall geometric design of the road are dealt with in ERA's *Geometric Design Manual-2002*. For trunk and link roads, carriageway widths of 7 meters or greater are be used throughout, and additional lanes will be needed when the capacity of a two-lane road is exceeded.

Shoulders participate in the structural function of a road pavement, providing lateral support for the pavement layers. They should help in removing surface water from the road surface and facilitate the internal drainage of the pavement. They are especially important when unbound materials are used in the pavement. From a functional point of view a minimum width of 1m is recommended and it is also recommended that shoulders on paved roads having a width less then 1m should be paved. Shoulders give additional width for emergency and temporary parking.

The main requirements for shoulders are their ability to support traffic on occasions, to be practically impervious and not prone to rapid erosion.

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

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

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

If gravel materials (unbound) are used for the construction of the shoulders, they should be of a quality similar to those described for subbase (see Section 6.2) or for gravel wearing courses (see Section 6.4).

For gravel roads, it is recommended that the shoulders be constructed with the same materials as the wearing course.

5.5 Typical Pavement Cross-Sections

Based on the above considerations, four alternative cross-sections are presented in Figure 5-1. It is to be noted that, unless the base course material is extended fully across the shoulders, some extra width is nevertheless provided for the base. This provides support to the edge of the pavement, where compaction is difficult to achieve. The extra width of the base course should be on the order of 20 to 30 cm. The edge seal covering the extra width of the base and the joint should extend a total of 40 to 60 cm.

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

Side drains should be avoided in areas with expansive soils. If side drains cannot be avoided due to site conditions, they shall be kept at a minimum distance of 4 - 6 m. from the toe of the embankment, dependent on the road functional classification. Side slopes shall also be flattened to 1:6 or flatter (see both *ERA Site Investigation Manual - 2002* and *ERA Geometric Design Manual - 2002* for further details). A more thorough discussion of expansive soils is given in Chapter 4.

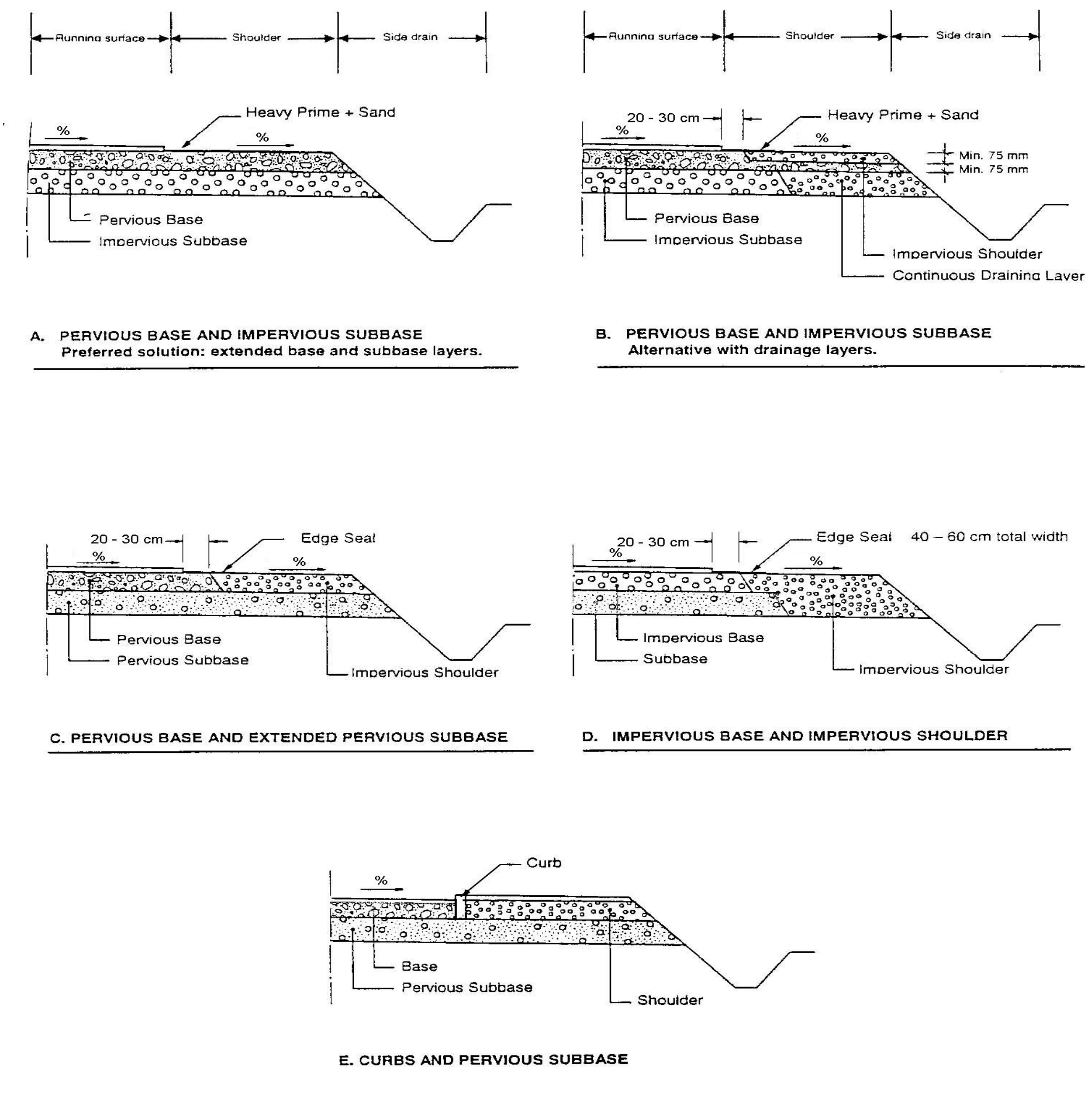


Figure 5-1: Typical Pavement Cross Sections

6. UNBOUND PAVEMENT MATERIALS

This chapter gives guidance on the selection of unbound materials for use as base course, sub-base, capping and selected subgrade layers. The main categories with a brief summary of their characteristics are shown in Table 6-1.

Table 6-1: Properties of Unbound Materials

Code	Description	Summary of Specification
GB1	Fresh, crushed rock	Dense graded, unweathered crushed stone,non-plastic parent fines
GB2	Crushed weathered rock, gravel or boulders	Dense grading, PI < 6, soil or parent fines
GB3	Natural coarsely graded granular material, including processed and modified gravels	Dense grading, PI < 6 CBR after soaking > 80
GS	Natural gravel	CBR after soaking > 30
GC	Gravel or gravel-soil	Dense graded; CBR after soaking > 15

Notes: 1. These specifications are sometimes modified according to site conditions, material type and principal use (see text).
2. GB = Granular base course, GS = Granular sub-base, GC = Granular capping layer.

6.1 Base Course Materials

A wide range of materials can be used as unbound base course including crushed quarried rock, crushed and screened, mechanically stabilized, modified or naturally occurring “as dug” or “pit run” gravels. Their suitability for use depends primarily on the design traffic level of the pavement and climate. However, all base course materials must have a particle size distribution and particle shape which provide high mechanical stability and should contain sufficient fines (amount of material passing the 0.425 mm sieve) to produce a dense material when compacted. In circumstances where several suitable types of base course materials are available, the final choice should take into account the expected level of future maintenance and the total costs over the expected life of the pavement. The use of locally available materials is encouraged, particularly at low traffic volumes (i.e. categories T1 and T2, see Table 2-6). Their use should be based on the results of performance studies and should incorporate any special design features which ensure their satisfactory performance.

Note: When considering the use of natural gravels a statistical approach should be applied in interpreting test results to ensure that their inherent variability is taken into account in the selection process.

For lightly trafficked roads the requirements set out below may be too stringent and in such cases reference should be made to specific case studies, preferably for roads under similar conditions.

6.1.1 CRUSHED STONE

Graded crushed stone (GB1). This material is produced by crushing fresh, quarried rock (GB1) and may be an all-in product, usually termed a 'crusher-run', or alternatively the material may be separated by screening and recombined to produce a desired particle size distribution, as per the specifications. Alternate gradation limits, depending on the local conditions for a particular project, are shown in Table 6-2. After crushing, the material should be angular in shape with a Flakiness Index (British Standard 812, Part 105) of less than 35%, and preferably of less than 30%. If the amount of fine aggregate produced during the crushing operation is insufficient, non-plastic angular sand may be used to make up the deficiency. In constructing a crushed stone base course, the aim should be to achieve maximum impermeability compatible with good compaction and high stability under traffic.

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

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

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

To ensure that the materials are sufficiently durable, they should satisfy the criteria given in Table 6-3. These are a minimum Ten Per Cent Fines Value (TFV) (British Standard 812, Part 111) and limits on the maximum loss in strength following a period of 24 hours of soaking in water. The likely moisture conditions in the pavement are taken into account in broad terms based on annual rainfall. Alternatively, requirements expressed in terms of the results of the Aggregate Crushing Value (ACV) (British Standard 812, Part 110) may be used: the ACV should preferably be less than 25 and in any case less than 29. Other simpler tests e.g. the Aggregate Impact Test (British Standard 812, Part 112, 1990**)** may be used in quality control testing provided a relationship between the results of the chosen test and the TFV has been determined. Unique relationships do not exist between the results of the various tests but good correlations can be established for individual material types and these need to be determined locally.

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

Typical Annual Rainfall (mm)	**Minimum 10% Fines Values (kN)**	**Minimum Ratio Wet/Dry Test (%)**
>500	110	75
<500	110	60

When dealing with materials originating from the weathering of basic igneous rocks, the recommendations in Section 6.1 *Naturally Occurring Granular Materials, Boulders, Weathered Rocks,* below, should be used.

The fine fraction of a GB1 material should be nonplastic.

These materials may be dumped and spread by grader but it is preferable to use a paver to ensure that the completed surface is smooth with a tight finish. The material is usually kept wet during transport and laying to reduce the likelihood of particle segregation.

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

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

6.1.2 NATURALLY OCCURRING GRANULAR MATERIALS, BOULDERS, WEATHERED ROCKS

Normal requirements for natural gravels and weathered rocks (GB2, GB3). A wide range of materials including lateritic, calcareous and quartzitic gravels, river gravels, boulders and other transported gravels, or granular materials resulting from the weathering of rocks can be used successfully as base course materials. Table 6-4 contains three recommended particle size distributions for suitable materials corresponding to maximum nominal sizes of 37.5 mm, 20 mm and 10 mm. Only the two larger sizes should be considered for traffic in excess of 1.5 million equivalent standard axles. To ensure that the material has maximum mechanical stability, the particle size distribution should be approximately parallel with the grading envelope.

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

Table 6-4: Recommended Particle Size Distributions for Mechanically Stable Natural Gravels and Weathered Rocks for Use as Base Course Material (GB2, GB3)

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

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

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

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

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

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

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

Arid and semi-arid areas. In Ethiopia, the low altitude areas of the northeast (low areas of Tigray, Welo and Hererge regions) and southeast (Hererge and Bale) are dry throughout most of the year. In these low rainfall areas, typically with a mean annual rainfall of less than 500 mm, and where evaporation is high, moisture conditions beneath a well sealed surface are unlikely to rise above the optimum moisture content determined in the ASTM Test Method D 1557 (Heavy Compaction). In such conditions, high strengths (CBR>80 %) are likely to develop even when natural gravels containing a substantial amount of plastic fines are used. In these situations, for the lowest traffic categories (Tl, T2) the maximum allowable PI can be increased to 12 and the minimum soaked CBR criterion reduced to 60% at the expected field density.

Materials of basic igneous origin. Materials in this group are sometimes weathered and may release additional plastic fines during construction or in service. Problems are likely to worsen if water enters the pavement and this can lead to rapid and premature failure. The state of decomposition also affects their long-term durability when stabilized with lime or cement. The group includes common rocks such as basalts and dolerites but also covers a wider variety of rocks and granular materials derived from their weathering, transportation or other alteration. Normal aggregate tests are often unable to identify unsuitable materials in this group. Even large, apparently sound particles may contain minerals that are decomposed and potentially expansive. The release of these minerals may lead to a consequent loss in bearing capacity. There are several methods of identifying unsound aggregates. These include petrographic analysis to detect secondary (clay) minerals and the use of various chemical soundness tests, e.g. sodium or magnesium sulphate (ASTM C 88). Indicative limits based on these tests include (a) a maximum secondary mineral content of 20%, (b) a maximum loss of 12 or 20% after 5 cycles in the sodium or magnesium sulphate tests respectively. In most cases it is advisable to seek expert advice when considering their use, especially when new deposits are being evaluated. It is also important to subject the material to a range of tests since no specific method can consistently identify problem materials.

In some areas of Ethiopia, weathered basalt gravels are available in large quantities. To study the performance of weathered basalt gravel, experimental roads were constructed in Ethiopia, namely on the Gelenso-Mechara project and Ghion-Jimma project under the Joint Road Research Project of the Ethiopian Transport Construction Authority and TRRL (Ref. 3).

Results to date indicate that these materials stabilized with 3 per cent of lime and surface dressed should provide an acceptable alternative to crushed stone base construction for main roads in Ethiopia. A particular advantage of this material is that it avoids the problem of clay working up into the base, which is a frequent source of failure when using crushed stone over active clay.

Materials of marginal quality. Naturally occurring gravels which do not normally meet the normal specifications for base course materials have occasionally been used successfully. They include lateritic, calcareous and volcanic gravels. In general their use should be confined to the lower traffic categories (i.e. T1 and T2) unless local studies have shown that they have performed successfully at higher levels.

Laterite gravels with plasticity index in the range of 6-12 and plasticity modulus in the range of 150-250 is recommended (Ref. 9) for use as base course material for T3 level of traffic volume. The values towards higher range are valid for semi-arid and arid areas of Ethiopia, i.e. with annual rainfall less than 500 mm.

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

Cinder gravels can also be used as a base course materials in lightly trafficked (T1 and T2) surface dressed roads (Ref. 4).

6.2 Sub-Bases (GS)

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

6.2.1 Bearing Capacity

A minimum CBR of 30 per cent is required at the highest anticipated moisture content when compacted to the specified field density, usually a minimum of 95 per cent of the maximum dry density achieved in the ASTM Test Method D 1557 (Heavy Compaction). Under conditions of good drainage and when the water table is not near the ground surface (see Chapter 3) the field moisture content under a sealed pavement will be equal to or less than the optimum moisture content in the ASTM Test Method D 698 (Light Compaction). In such conditions, the sub-base material should be tested in the laboratory in an unsaturated state. Except in arid areas (Category (3) in Chapter 3), if the base course allows water to drain into the lower layers, as may occur with unsealed shoulders and under conditions of poor surface maintenance where the base course is pervious (see Section 3.1), saturation of the sub-base is likely. In these circumstances, the bearing capacity should be determined on samples soaked in water for a period of four days. The test should be conducted on samples prepared at the density and moisture content likely to be achieved in the field. In order to achieve the required bearing capacity, and for uniform support to be provided to the upper pavement, limits on soil plasticity and particle size distribution may be required. Materials which meet the recommendations of Tables 6-5 and 6-6 will usually be found to have adequate bearing capacity.

6.2.2 Use as a Construction Platform

In many circumstances the requirements of a sub-base are governed by its ability to support construction traffic without excessive deformation or ravelling. A high quality sub-base is therefore required where loading or climatic conditions during construction are severe. Suitable material should possess properties similar to those of a good surfacing material for unpaved roads. The material should be well graded and have a plasticity index at the lower end of the appropriate range for an ideal unpaved road wearing course under the prevailing climatic conditions. These considerations form the basis of the criteria given in Tables 6-5 and 6-6. Material meeting the requirements for severe conditions will usually be of higher quality than the standard sub-base (GS). If materials to these requirements are unavailable, trafficking trials should be conducted to determine the performance of alternative materials under typical site conditions.

In the construction of low-volume roads, where cost savings at construction are particularly important, local experience is often invaluable and a wider range of materials may often be found to be acceptable.

In Ethiopia, laterite is one of the widely available material and can be used as a sub-base material. Laterite meeting the gradation requirements of Table 6-6 can be used for traffic levels up to $3x10^6$ ESA provided the following criteria is satisfied (Ref. 9):

Plasticity Index (%)	< 25
Plasticity Modulus (PM)	< 500
CBR (%)	> 30

Table 6-5: Recommended Plasticity Characteristics for Granular Sub-Bases (GS)

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

**Table 6-6: Typical Particle Size Distribution for Sub-Bases (GS)
Which Will Meet Strength Requirements**

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

6.2.3 Sub-Base as a Filter or Separating Layer

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

The following criteria should be used to evaluate a subbase as a separating or filter layer:

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

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

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

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

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

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

6.3 Selected Subgrade Materials and Capping Layers (GC)

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

As an illustrative example, approximately 30 cm of “GC” material (as described below) placed on an S1 or S2 subgrade will allow to select a pavement structure as for an S3 subgrade. An additional 5 cm of “GC” material may allow to consider an S4 subgrade class.

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

7. CEMENT AND LIME STABLIZED MATERIALS

7.1 Introduction

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

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

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

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

- A substantial proportion of their strength is retained when they become saturated with water.
- Surface deflections are reduced.
- Materials in the supporting layer cannot contaminate the stabilized layer.
- Lime-stabilized material is suitable for use as a capping layer or working platform when the in situ material is excessively wet or weak and removal is not economical.

Associated with these desirable qualities are several possible problems:

- Traffic, thermal and shrinkage stresses can cause stabilized layers to crack.
- Cracks can reflect through the surfacing and allow water to enter the pavement structure.
- If carbon dioxide has access to the material, the stabilization reactions are reversible and the strength of the layers can decrease.
- The construction operations require more skill and control than for the equivalent unstabilized material.

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

The minimum acceptable strength of a stabilized material depends on its position in the pavement structure and the level of traffic. It must be sufficiently strong to resist traffic stresses but upper limits of strength are usually set to minimize the risk of reflection cracking. Three types of stabilized layer have been used in the structural design catalog (Chapter 10) and the strengths required for each are defined in Table 7-1.

Table 7-1: Properties of Cement and Lime-Stabilized Materials

Code	Description	Unconfined compressive strength* (Mpa) (Cement Stabilized)	Minimum CBR value* (%) (Lime stabilized)
CB1	Stabilized base course	3.0 - 6.0	100
CB2	Stabilized base course	1.5 - 3.0	80
CS	Stabilized sub-base	0.75 - 1.5	40

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

7.2 The Stabilization Process

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

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

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

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

7.3 Selection of Type of Treatment

The selection of the stabilizer is based on the plasticity and particle size distribution of the material to be treated. The appropriate stabilizer can be selected according to the criteria shown in Table 7-2:

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

PP ≤60

Type of stabilization	Soil properties					
	More than 25% passing the 0.075 mm sieve			*Less than 25% passing the 0.075 mm sieve*		
	PI ≤10	10<PI ≤20	PI>20	PI ≤6	PI ≤10	PI > 10
Cement	Yes	Yes	*	Yes	Yes	Yes
Lime	*	Yes	Yes	No	*	Yes
Lime-Pozzolan	Yes	*	No	Yes	Yes	*

Notes. 1. * Indicates that the agent will have marginal effectiveness
2. PP = Plasticity Product (see Chapter 6).

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

If possible, the quality of the material to be stabilized should meet the minimum standards set out in Table 7-3. Stabilized layers constructed from these materials are more likely to perform satisfactorily even if they are affected by carbonation during their lifetime. Materials not complying with Table 7-3 can sometimes be stabilized but more additive will be required and the cost and the risk from cracking and carbonation will increase.

Table 7-3: Desirable Properties of Material before Stabilization

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

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

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

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

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

7.4 Cement Stabilization

7.4.1 Selection of Cement Content

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

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

7.4.2 Preparation of Specimens

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

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

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

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

7.5 Lime Stabilization

7.5.1 Properties of Lime-Stabilized Materials

By lime-stabilization, both the ion exchange reaction and the production of cementitious materials increase the stability and reduce the volume change within the clay fraction. It is not unusual for the swell to be reduced from 7 or 8 per cent to 0.1 per cent by the addition of lime. The ion exchange reaction occurs quickly and can increase the CBR of clayey materials by a factor of two or three.

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

7.5.2 TYPES OF LIME

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

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

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

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

In many parts of the world, lime has been produced on a small scale for many hundreds of years to make mortars and lime washes for buildings. Different types of kilns have been used and most appear to be relatively effective. Trials have been carried out by TRRL in Ghana (Ref. 11) to determine the output possible from small kilns and to assess the suitability of lime produced without commercial process control for soil stabilization. Small batch kilns have subsequently been used to produce lime for stabilized layers on major road projects.

7.5.3 SELECTION OF LIME CONTENT

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

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

8. BITUMEN-BOUND MATERIALS

8.1 Introduction

This chapter describes types of bituminous materials, commonly referred to as premixes, which are manufactured in asphalt mixing plants and laid hot (hence the other used designation, “hot-mix”). In-situ mixing can also be used for making base courses for lower standard roads but these methods are not generally recommended and are not discussed in detail here.

Note: This chapter is not intended to replace standard specifications, to which the designer should refer. Rather, it is intended to outline the basic qualities assumed in the development of the catalog of pavement structures (Chapter 10) and aid the designer in making choices in the formulation of supplementary specifications tailored to the specific conditions and availability of materials of a specific project.

8.2 Components of a Mix

The coarse aggregates for premixes should be produced by crushing sound, unweathered rock or natural gravel. The specifications for the aggregates are similar to those for granular base courses. The aggregate must be clean and free of clay and organic material, the particles should be angular and not flaky. Gravel should be crushed to produce at least two fractured faces on each particle. Aggregates for wearing course must also be resistant to abrasion and polishing. Highly absorptive aggregates should be avoided where possible, but otherwise the absorption of bitumen must be taken into account in the mix design procedure. Hydrophilic aggregates which have a poor affinity for bitumen in the presence of water should also be avoided. They may be acceptable only where protection from water can be guaranteed.

The fine aggregate can be crushed rock or natural sand and should also be clean and free from organic impurities. The **filler** (material passing the 0.075 mm sieve) can be crushed rock fines, Portland cement or hydrated lime. Portland cement or hydrated lime is often added to natural filler (1-2 % by mass of total mix) to assist the adhesion of the bitumen to the aggregate. Fresh hydrated lime can help reduce the rate of hardening of bitumen in surface dressings and may have a similar effect in premixes.

Suitable specifications for the coarse and fine mineral components are given in Tables 8-1 and 8-2.

8.3 Bituminous Surfacings

The highest quality material is necessary for the bituminous surfacing. Where thick bituminous surfacings are required, they are normally constructed with a wearing course laid on a binder course (see Figure 1-2) which can be made to slightly less stringent specifications.

Table 8-1: Coarse Aggregate for Bituminous Mixes

Property	Test	Specification
Cleanliness Decantation[1]	Sedimentation or 0.075mm sieve	< 5 per cent passing
Particle shape	Flakiness index[2]	< 45 per cent
Strength Value (ACV)[3]	Aggregate Crushing aggregates the Ten per Cent Fines Value Test (TFV) is used	< 25. For weaker
Aggregate Impact Value (AIV)[3]	< 25	
Los Angeles Abrasion Value (LAA)[4]	< 30 (wearing course) < 35 (other)	
Abrasion Value (AAV)[3]	Aggregate Abrasion < 12 (very heavy traffic)	< 15
Polishing (wearing course only)	Polished Stone Value[3]	Not less than 50-75 depending on location
Durability Sodium Test Magnesium Test	Soundness:[5] < 18 per cent	< 12 per cent
Water Absorption	Water Absorption[6]	< 2 per cent
Bitumen Affinity	Coating and Stripping[7] aggregate > 95 per cent	Non-stripped area of

Notes:
1. BS 812, Part 103
2. BS 812, Part 105
3. BS 812, Part 110
4. ASTM C 131 and C535
5. ASTM C 88
6. ASTM C 127
7. AASHTO T-182

Table 8-2: Fine Aggregate for Bituminous Mixes

Property	Test	Specification		
Cleanliness	Sedimentation or Decantation[1]	Per cent passing 0.075 mm sieve: Wearing courses: < 8% for sand fines < 17% for crushed rock fines Other Layers: <22%		
	Sand Equivalent[2] (material passing 4.75 mm sieve)	Traffic	Wearing course	Base course
		Light (<T3)	>35%	>45%
		Medium/Heavy	>40%	>50%
	Plasticity Index[3] (material passing 0.425 mm sieve)	< 4		
Durability	Soundness Test[4] (5 cycles)	Magnesium: < 20 per cent Sodium: < 15 per cent		

Note (1) See notes to Table 8-1
(2) AASHTO T 176
(3) ASTM D 4318
(4) ASTM C 88

To perform satisfactorily as road surfacings, bitumen aggregate mixes need to possess the following characteristics:

- High resistance to deformation.
- High resistance to fatigue and the ability to withstand high strains i.e. they need to be flexible.
- Sufficient stiffness to reduce the stresses transmitted to the underlying pavement layers.
- High resistance to environmental degradation i.e. good durability.
- Low permeability to prevent the ingress of water and air.
- Good workability to allow adequate compaction to be obtained during construction

The requirements of a mix which will ensure each of these characteristics are often conflicting. For example, mixes suitable for areas carrying heavy, slow-moving traffic, such as on climbing lanes, or areas where traffic is highly channeled, will be unsuitable for flat, open terrain where traffic moves more rapidly. A mix suitable for the latter is likely to deform on a climbing lane and a mix suitable for a climbing lane is likely to possess poor durability in flat terrain. In severe locations the use of bitumen modifiers is often advantageous.

It has been shown that 40/50, 60/70 and 80/100 penetration grade bitumens in the surface of wearing courses all tend to harden to a similar viscosity within a short time. It is therefore recommended that 60/70 penetration bitumen is used to provide a suitable compromise between workability, deformation resistance and potential hardening in service. If possible, a bitumen should be selected which has a low temperature sensitivity and good resistance to hardening as indicated by the standard and extended forms of the Rolling Thin Film Oven Test (ASTM D 2872).

8.4 Common Types of Premix

The main types of premix are asphaltic concrete, bitumen macadam and hot rolled asphalt. Each type can be used in surfacings or base courses. Their general properties and suitable specifications described below.

8.4.1 Asphaltic Concrete

Asphaltic concrete (AC) is a dense, continuously graded mix which relies for its strength on both the interlock between aggregate particles and, to a lesser extent, on the properties of the bitumen and filler. The mix is designed to have low air voids and low permeability to provide good durability and good fatigue behavior but this makes the material particularly sensitive to errors in proportioning, and mix tolerances are therefore very narrow.

The particle size distributions for wearing course material given in Table 8-3 have produced workable mixes that have not generally suffered from deformation failures.

Table 8-3: AsphalticConcrete Surfacings

Mix designation	WC1	WC2	BC1
	Wearing course		*Binder Course*
Test sieve (mm)	Percentage by mass of total aggregate passing test sieve		
28	-	-	100
20	100	-	80 – 100
10	60 – 80	100	60 – 80
5	54 – 72	62 – 80	36 – 56
2.36	42 – 58	44 – 60	28 – 44
1.18	34 – 48	36 – 50	20 – 37
0.6	26 – 38	28 – 40	15 – 27
0.3	18 – 28	20 – 30	10 – 20
0.15	12 – 20	12 – 20	5 – 13
0.075	6 – 12	6 – 12	2 – 6
Bitumen content (1) (per cent by mass of total mix)	5.0 – 7.0	5.5 - 7.4	4.8 - 6.1
Bitumen grade (pen)	60/70 or 80/100	60/70 or 80/100	60/70 or 80/100
Thickness (2) (mm)	40 – 50	30 - 40	50 – 65

Notes. 1. Determined by Marshall design method (ASTM D1559)
2. In practice the upper limit has been exceeded by 20% with no adverse effect

It is common practice to design the mix using the Marshall Test (ASTM D1559) and to select the design binder content by calculating the mean value of the binder contents for (a) maximum stability, (b) maximum density, (c) the mean value for the specified range of void contents and (d) the mean value for the specified range of flow values. Compliance of properties at this design binder content with recommended Marshall criteria is then obtained (Table 8-4).

Table 8-4: Suggested Marshall Test Criteria

Total Traffic (10^6 ESA)	< 1.5	1.5 - 10.0	> 10.0
Traffic classes	Tl,T2,T3	T4,T5,T6	T7,T8
Minimum stability (kN at 60^0C)	3.5	6.0	7.0
Minimum flow (mm)	2	2	2
Compaction level (Number of blows)	2 x 50	2 x 75	2 x 75
Air voids (per cent)	3 - 5	3 - 5	3 – 5

A good method of selecting the Marshall design binder content is to examine the range of binder contents over which each property is satisfactory, define the common range over which all properties are acceptable, and then choose a design value near the center of the common range. If this common range is too narrow, the aggregate grading should be adjusted until the range is wider and tolerances less critical.

To ensure that the compacted mineral aggregate in continuously graded mixes has a voids content large enough to contain sufficient bitumen, a minimum value of the voids in the mineral aggregate (VMA) is specified, as shown in Table 8-5.

Table 8-5: Voids in the Mineral Aggregate

Nominal maximum particle size(mm)	Minimum voids in mineral aggregate (per cent)
37.5	12
28	12.5
20	14
144	15
10	16
5	18

8.4.2 Bitumen Macadam

Close graded bitumen macadams are continuously graded mixes similar to asphaltic concretes but usually with a less dense aggregate structure. They have been developed in the United Kingdom from empirical studies and are made to recipe specifications without reference to a formal design procedure. Their suitability for different conditions and with different materials may be questioned but, in practice, numerous materials including crushed gravels have been used successfully. The advantage of this method is that quality control testing is simplified and this should allow more intensive compliance testing to be performed. Aggregates which behave satisfactorily in asphaltic concrete will also be satisfactory in dense bitumen macadam. Suitable specifications for a base course mix are given in Table 8-6.

Table 8-6: Bitumen Macadam

Mix designation	BC2
	Binder Course
Test sieve (mm)	Percentage by mass of total aggregate passing test sieve
28	100
20	95 – 100
14	65 – 85
10	52 – 72
6.3	39– 55
3.35	32 – 46
1.18	-
0.3	7 – 21
0.075 [(1)]	2 – 8
Bitumen content [(2)] (per cent by mass of total mix)	5.0 ± 0.6
Bitumen grade [(3)] (pen)	60/70 or 80/100
Thickness [(4)] (mm)	50 – 80

Notes:
1. When gravel other than limestone is used, the anti-stripping properties will be improved by including 2% Portland cement or hydrated lime in the material passing the 0.075 mm sieve.
2. For aggregate with fine microtexture e.g. limestone, the bitumen content should be reduced by 0.1 to 0.3%.
3. 60/70 grade bitumen is preferred, see text.
4. In practice the upper limit has been exceeded by 20% with no adverse effect.
5. Limestone and gravel are not recommended for wearing courses where high skidding resistance is required.

Close graded bitumen macadam mixes offer a good basis for the design of deformation resistant materials for severe sites, and in these cases they should be designed on the basis of their refusal density (see paragraph below). Recipe mixes are not recommended in these circumstances are the Marshall design criteria in Table 8-7 should be used. At the time of construction the air voids content is virtually certain to be in excess of five per cent and therefore a surface dressing should be placed soon after construction.

Table 8-7: Suggested Marshall Criteria for Close Graded Bitumen Macadams

Design Traffic (10^6 ESA)	< 1.5	1.5 - 10.0	> 10.0	Severe Sites
Traffic classes	Tl,T2,T3	T4,T5,T6	T7,T8	-
Minimum stability (kN at 60^0C)	3.5	6.0	7.0	9.0
Minimum flow (mm)	2-4	2-4	2-4	2-4
Compaction level (Number of blows)	2 x 50	2 x 75	To refusal	To refusal

8.4.3 Bituminous Surfacing

It is essential that the thin bituminous surfacings (50mm) recommended for structures described in Charts 3,4 and 7 of the structural catalog are flexible. This is particularly important for surfacings laid on granular base courses. Mixes which are designed to have good durability rather than high stability are flexible and are likely to have “sand” and bitumen contents at the higher end of the permitted ranges. In areas where the production of sand-sized material is expensive and where there is no choice but to use higher stability mixes, additional stiffening through the aging and embrittlement of the bitumen must be prevented by applying a surface dressing.

8.4.4 Design to Refusal Density

Under severe loading conditions asphalt mixes must be expected to experience significant secondary compaction in the wheel paths. Severe conditions cannot be precisely defined but will consist of a combination of two or more of the following:

- High maximum temperatures
- Very heavy axle loads
- Very channeled traffic
- Stopping or slow moving heavy vehicles

Failure by plastic deformation in continuously graded mixes occurs very rapidly once the VIM are below 3 per cent. Therefore the aim of refusal density design is to ensure that at refusal there is still at least 3 per cent voids in the mix.

For sites which do not fall into the severe category, the method can be used to ensure that the maximum binder content for good durability is obtained. This may be higher than the Marshall optimum but the requirements for resistance to deformation will be maintained. Where lower axle loads and higher vehicle speeds are involved, the minimum VIM at refusal can be reduced to 2 per cent.

Refusal density can be determined by two methods:

(a) Extended Marshall Compaction
(b) Compaction by vibrating hammer

Details of the tests and their limitations are given in Appendix D.

8.5 Bituminous Base Courses

Satisfactory bituminous base courses can be made using a variety of specifications. They should possess properties similar to bituminous mix surfacings but whenever they are used in conjunction with such a surfacing the loading conditions are less severe, hence the mix requirements are less critical. Nevertheless, the temperatures of base courses may be high and the mixes are therefore prone to deformation in early life, and aging and embrittlement later.

8.5.1 Principal Mix Types

Particle size distributions and general specifications for continuously graded mixes are given in Table 8-8. No formal design method is generally available for determining the optimum composition for these materials because the maximum particle size and proportions of aggregate greater than 25 mm precludes the use of the Marshall Test.

Table 8-8: Bitumen Macadam Base Course

Mix designation	RB1
Test sieve (mm)	Percentage by mass of total aggregate passing test sieve
50	100
37.5	95 – 100
28	70 – 94
14	56 – 76
10	44 – 60
5	32 – 46
0.3	7 – 21
0.075	2 – 8 (1)
Bitumen content (per cent by mass of total mix)	4.0 (2) ± 0.5
Thickness (mm)	65 – 125
Voids (per cent)	4 – 8
Bitumen grade (pen)	60/70 or 80/100

Notes. 1.Where gravel other than limestone is used, the anti-stripping properties will be improved by including 2% Portland cement or hydrated lime in the material passing the 0.075 mm sieve.
2. Up to 1% additional bitumen may be required for gravel aggregate.

These specifications are recipes which have been developed from experience and rely on performance data for their optimum adaptation to local conditions. The following principles should be adopted for all bituminous layers but are particularly important for recipe type specifications:

(i) Trials for mix production, laying and compaction should be carried out to determine suitable mix proportions and procedures.
(ii) Durable mixes require a high degree of compaction and this is best achieved by specifying density in terms of maximum theoretical density of the mix.
(iii) Mixing times and temperatures should be set at the minimum required to achieve good coating of the aggregates and satisfactory compaction.
(iv) The highest bitumen content commensurate with adequate stability should be used.

8.5.2 Sand-Bitumen Mixes

For light and medium trafficked roads (defined as roads carrying less than 300 commercial vehicles per day and with mean equivalent standard axles per vehicle of 0.5 or less) and in areas lacking coarse aggregates, bitumen stabilized sands are an alternative. Best results are achieved with well-graded angular sands in which the proportion of material passing the 0.075mm sieve does not exceed 10% and is non-plastic. The bitumen can range from a viscous cutback that will require heating to a more fluid cutback or emulsion that can be used at ambient temperatures. The most viscous cutbacks that can be properly mixed at ambient temperatures are RC or MC 800 or equivalents. In general, the more viscous the bitumen the higher will be the stability of the mix.

The amount of bitumen required will generally lie between 3 and 6 per cent by weight of the dry sand, the higher proportions being required with the finer-grained materials.

The Marshall Test (ASTM D1559) can be used for determining the amount of bitumen required (ref. 12). Design criteria are given in Table 8-9 for sand bitumen mixes used as base course materials for tropical roads carrying medium to light traffic.

Table 8-9: Criteria for Sand-Bitumen Base Course Materials

	Traffic Classes	
	Tl	T2
Marshall stability at 60^{o}C (min)	1 kN	1.5 kN
Marshall flow value at 60^{o}C (max)	2.5 mm	2 mm

8.6 Manufacture and Construction

General guidance on the design, manufacture and testing of bitumen macadams can be found in the British Standards, BS 4987. Similar guidance for asphalt concrete is given in the publications of the Asphalt Institute (Refs. 12-14).

It is normal practice to carry out preliminary design testing to determine the suitability of available aggregates and their most economical combination to produce a job mix formula.

The importance of detailed compaction trials at the beginning of asphalt construction work cannot be over emphasized. During these trials, compaction procedures and compliance of the production-run asphalt with the job-mix formula should be established. Adjustments to the job-mix formula and, if necessary, redesign of the mix are carried out at this stage to ensure that the final job mix satisfies the mix design requirements and can be consistently produced by the plant.

Tolerances are specified for bitumen content and for the aggregate grading to allow for normal variation in plant production and sampling. Typical tolerances for single tests are given in Table 8-10. Good quality control is essential to obtain durable asphalt and the mean values for a series of tests should be very close to the job-mix formula which, in turn, should have a grading entirely within the specified envelope.

Mixing must be accomplished at the lowest temperatures and in the shortest time that will produce a mix with complete coating of the aggregate and at a suitable temperature to ensure proper compaction. The ranges of acceptable mixing and rolling temperatures are shown in Table 8-11. Very little additional compaction is achieved at the minimum rolling temperatures shown in the table and only pneumatic tired rollers should be used at these temperatures.

Table 8-10: Job-Mix Tolerances for a Single Test

Combined aggregate passing test sieve (mm)		Bitumen content	
Test sieve	Per cent	Mix type	Per cent
12.5+	± 5	Wearing Courses	± 0.3
10.0	± 5		
2.36	± 5	Binder Courses	± 0.4
0.60	± 4		
0.30	± 3	Base Courses	± 0.4
0.15	+2		
0.075	+2		

Table 8-11: Manufacturing and Rolling Temperatures (^{o}C)

Grade of bitumen (pen)	Bitumen Mixing	Aggregate Mixing	Mix Rolling (minimum)
80 – 100	130 - 160	130 – 155	80
60 – 70	150 – 175	150 - 170	90
40 – 50	160 - 175	160 - 170	100

Rolled asphalts are relatively easy to compact but bitumen macadams and asphaltic concretes are relatively harsh and more compactive effort is required. Heavy pneumatic tired rollers are usually employed, the kneading action of the tires being important in orientating the particles. Vibratory compaction has been used successfully but care is needed in selecting the appropriate frequency and amplitude of vibration, and control of mix temperature is more critical than with pneumatic tired rollers. Steel-wheeled dead-weight rollers are relatively inefficient and give rise to a smooth surface with poor texture but are required to obtain satisfactory joints. Rolling usually begins near the shoulder and progresses towards the center. It is important that directional changes of the roller are made only on cool compacted mix and that each pass of the roller should be of slightly different length to avoid the formation of ridges. The number of joints to cold, completed edges should be minimized by using two pavers in echelon of a full-width paver to avoid cold joints between adjacent layers. If this is not possible, repositioning of the paver from lane to lane at frequent intervals is another option.

If a layer is allowed to cool before the adjacent layer is placed, then the edge of the first layer must be “roller over” and thoroughly compacted. Before laying the second lane, the cold joint should be broomed if necessary and tack coated.

The paver screed should be set to overlap the first mat by a sufficient amount to allow the edge of the rolled over layer to be brought up to the correct level. Coarse aggregates in the material overlapping the cold joint should be carefully removed. The remaining fine materials will allow a satisfactory joint to be constructed.

9. SURFACE TREATMENTS

9.1 Introduction

This chapter presents a general guide to the design of surface treatments and draws attention to some of the more common mistakes that are made. It provides a framework on which the engineer can base specific decisions to suit particular local conditions, thereby producing cost effective results. It also contains brief descriptions of certain other types of surface treatments.

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

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

A surface treatment can provide an effective and economical running surface for newly constructed road pavements. For sealing new roadbases, traffic flows of up to 500 vehicles/lane/day are appropriate, although this can be higher if the roadbase is very stable or if a triple seal is used. Roads carrying in excess of 1000 vehicles/lane/day, have been successfully surfaced with multiple surface treatments.

A correctly designed and constructed surface treatment should last at least 5 years before resealing with another surface treatment becomes necessary. If traffic growth over a period of several years necessitates a more substantial surfacing or increased pavement thickness, a bituminous overlay can be laid over the original surface treatment when the need arises.

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

9.2 Types of Surface Treatment

Surface treatments can be constructed in a number of ways to suit site conditions. The common types of surface treatments are illustrated in Figure 9-1.

9.2.1 SINGLE SURFACE TREATMENT

A single surface treatment would not normally be used on a new roadbase because of the risk that the film of bitumen will not give complete coverage. It is also particularly important to minimize the need for future maintenance and a double dressing should be considerably more durable than a single dressing. However, a ‘racked-in’ dressing (see below) may be suitable for use on a new roadbase which has a tightly knit surface because of the heavier applications of binder which is used with this type of single dressing.

When applied as a maintenance operation to an existing bituminous road surface a single surface treatment can fulfill the functions required of a maintenance re-seal, namely waterproofing the road surface, arresting deterioration, and restoring skid resistance.

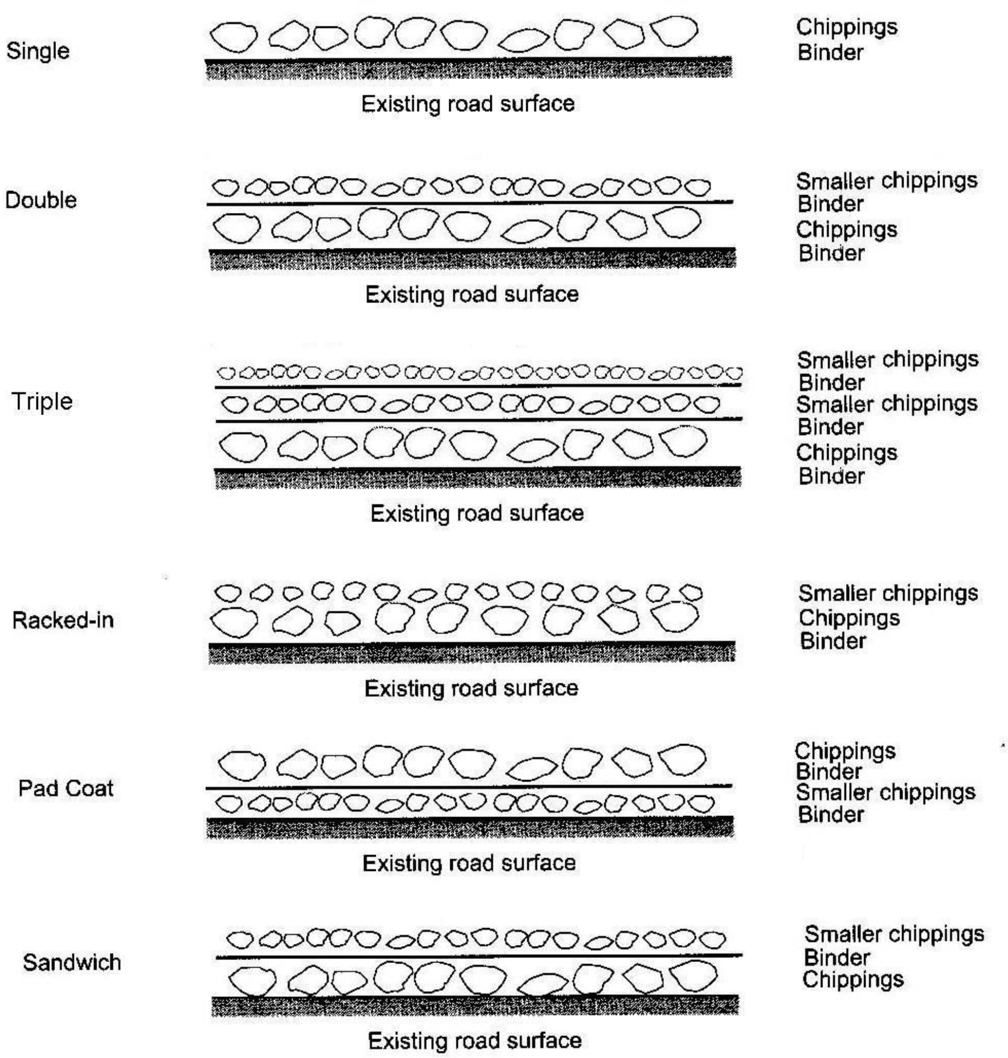


Figure 9-1: Types of Surface Treatments

9.2.2 DOUBLE SURFACE TREATMENT

Double surface treatments should be used when:

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

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

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

9.2.3 Triple Surface Treatment

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

9.2.4 Racked-In Surface Treatment

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

- Less risk of dislodged large chippings.
- Early stability through good mechanical interlock.
- Good surface texture.

9.2.5 Other Types of Surface Treatment

'Pad coats' are used where the hardness of the existing road surface allows very little embedment of the first layer of chippings, such as on a newly constructed cement stabilized roadbase or a dense crushed rock base. A first layer of nominal 6mm chippings will adhere well to the hard surface and will provide a 'key' for larger 10mm or 14mm chippings in the second layer of the treatment.

'Sandwich' surface treatments are principally used on existing binder rich surfaces and sometimes on gradients to reduce the tendency for the binder to flow down the slope.

9.3 Chippings for Surface Treatments

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

It is recommended that chippings used of surface treatment should comply with the requirements of Table 9-1 for higher levels of traffic, and to the requirements of Table 9-2 for lightly trafficked roads of up to 250 vehicles per day:

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

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

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

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

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

Specifications for maximum aggregate crushing value (ACV) for surface treatment chippings typically lie in the range 20 to 35. For lightly trafficked roads the higher value is likely to be adequate but on more heavily trafficked roads a maximum ACV of 20 is recommended.

The polished stone value (PSV) of the chippings is important if the primary purpose of the surface treatment is to restore or enhance the skid resistance of the road surface. The PSV required in a particular situation is related to the nature of the road site and the speed and intensity of the traffic (Ref. 15). The resistance to skidding is also dependent upon the macro texture of the surface which, in turn, is affected by the durability of the exposed aggregate. Table 9-3 gives recommended values of PSV for various road and traffic conditions and provides and indication of the required aggregate properties.

Table 9-3: Recommended Polished Stone Values of Chippings

<table>
<tr><th></th><th colspan="14">Traffic (cv/l/d) at Design Life</th></tr>
<tr><th>Site Definition</th><th>0 to 100</th><th>101 to 250</th><th>251 to 500</th><th>501 to 750</th><th>751 to 1000</th><th>1001 to 1250</th><th>1251 to 1500</th><th>1501 to 1750</th><th>1751 to 2000</th><th>2002 to 2250</th><th>2251 to 2500</th><th>2501 to 2750</th><th>2751 to 3250</th><th>Over 3250</th></tr>
<tr><td>1 Dual carriageway non-event sections and minor junctions</td><td colspan="8">55</td><td colspan="2">57</td><td colspan="2">60</td><td>65</td><td>68</td></tr>
<tr><td>2 Single carriageway non-event sections and minor junctions</td><td>45</td><td>50</td><td colspan="2">53</td><td>55</td><td colspan="2">57</td><td>60</td><td colspan="2">63</td><td colspan="2">65</td><td colspan="2">68</td></tr>
<tr><td>3 Approaches to and across major junctions(all limbs)

Gradient 5%-10%, Longer than 50m

Bend, radius 100-250m.

Roundabout</td><td>50</td><td>55</td><td>57</td><td colspan="2">60</td><td colspan="2">63</td><td colspan="2">65</td><td colspan="2">68</td><td colspan="3">Over 70</td></tr>
<tr><td>4 Gradient > 10%, longer than 50m

Bend,radius < 100m</td><td>55</td><td>60</td><td colspan="2">63</td><td colspan="2">65</td><td colspan="2">68</td><td colspan="3">Over 70</td><td colspan="3" rowspan="2"></td></tr>
<tr><td>5 Approach to roundabout, traffic signals, pedestrian crossing, railway level crossing, etc.</td><td>63</td><td>65</td><td colspan="2">68</td><td colspan="7">Over 70</td></tr>
</table>

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

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

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

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

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

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

- Spraying the chippings with a light application of creosote, diesel oil, or kerosene at ambient temperature (Ref. 16). This can be conveniently done as the chippings are transferred from stockpile to gritting lorries by a belt conveyor or, alternatively, they can be mixed in a simple concrete mixer.

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

Chippings which are pre-coated with bitumen enable the use of a harder grade of binder for construction which can provide early strong adhesion and thus help to obtain high quality dressings. The binder used for pre-coating need not necessarily be the same kind as that used for the surface treatment; for example, tar-coated chippings adhere well to a sprayed bitumen film. Pre-coating is usually undertaken in a hot-mix plant and the hardness of the coating, and thus the tendency for the chippings to adhere to each other, can be controlled by the mixing temperature and/or the duration of mixing; typical coating temperatures are about 140^{o}C for bitumen binders and 120^{o}C for tar binders. Table 9-4 indicates the amount of binder recommended for lightly coating chippings.

Table 9-4: Binder Contents for Lightly-Coated Chippings

Nominal Size of Chippings (mm)	***Target Binder Content (per cent by mass)***	
	Bitumen	*Tar*
6	1.0	1.2
10	0.8	1.0
14	0.6	0.8
20	0.5	0.7

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

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

9.4 Bitumens

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

9.4.1 Prime Coats

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

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

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

Low viscosity, medium curing cutback bitumens such as MC-30, MC-70, or in rare circumstances MC-250, can be used for prime coats (Ref. 17). The relationship between grade and viscosity (see Appendix A) for cutback primes is shown in Table 9-5.

Table 9-5: Kinematic Viscosities of Current Cutback Binders

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

9.4.2 Bitumens for Surface Treatments

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

- be capable of being sprayed;
- 'wet' the surface of the road in a continuous film;
- not run off a cambered road or form pools of binder in local depressions;
- 'wet' and adhere to the chipping at road temperature;
- be strong enough to resist traffic forces and hold the chippings at the highest prevailing ambient temperatures;
- remain flexible at the lowest ambient temperature, neither cracking nor becoming brittle enough to allow traffic to 'pick-off' the chippings; and
- resist premature weathering and hardening.

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

Figure 9-2 shows the permissible range of binder viscosity for successful surface treatment at various road surface temperatures. In Ethiopia, daytime road temperatures typically lie between about 25°C and 50°C, normally being in the upper half of this range unless heavy rain is falling. For these temperatures the viscosity of the binder should lie between approximately 10^4 and $7x10^5$ centistokes. At the lower road temperatures cutback grades of bitumen are most appropriate, whilst at higher road temperatures penetration grade bitumens can be used.

The temperature/viscosity relationships shown in Figure 9-2 do not apply to bitumen emulsions. These have a relatively low viscosity and 'wet' the chippings readily, after which the emulsion 'breaks,' the water evaporates and particles of high viscosity bitumen adhere to the chippings and the road surface.

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

- Penetration grade
- Emulsion
- Cutback
- Modified bitumens

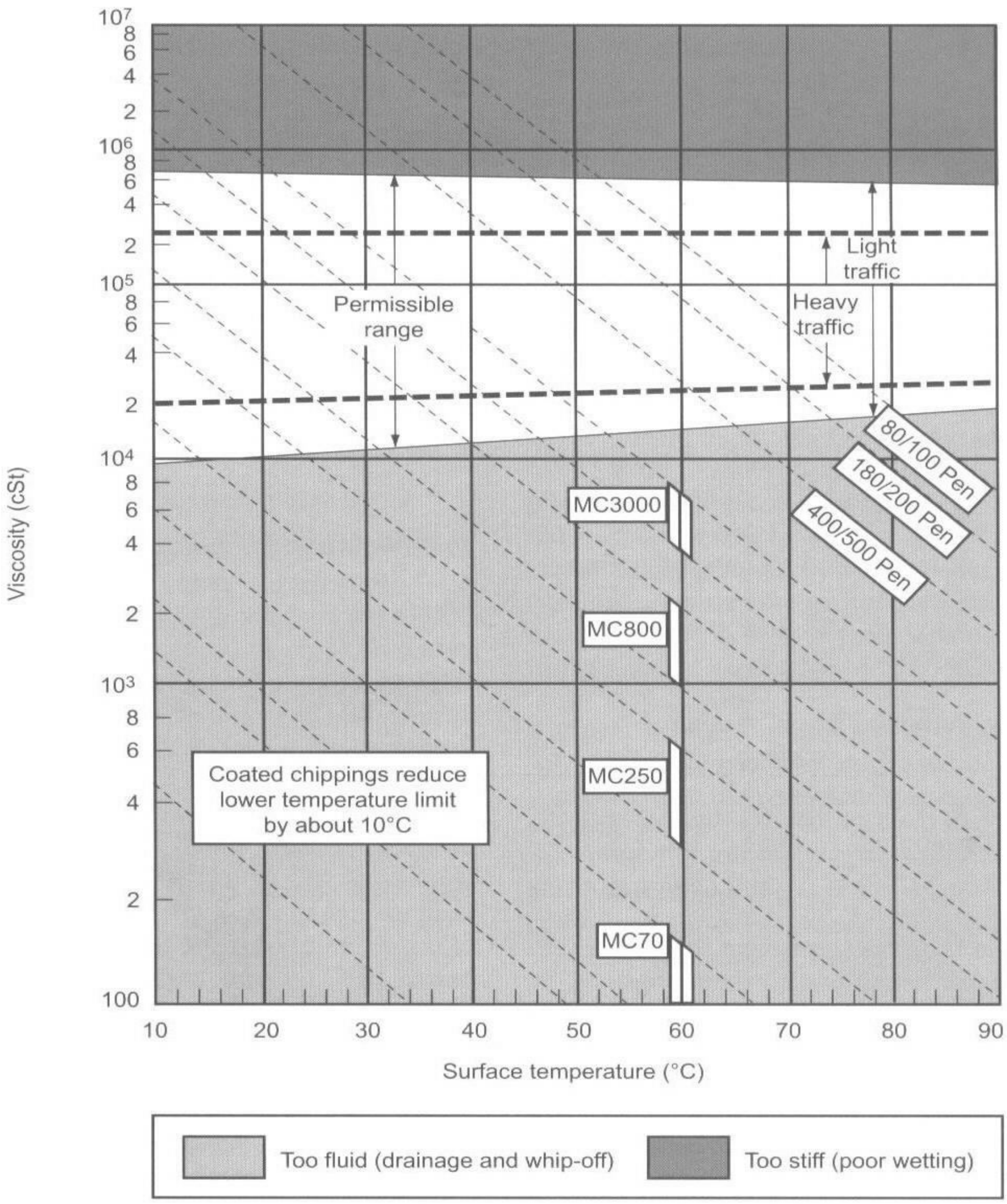


Figure 9-2: Surface Temperature/Choice of Binder for Surface Treatments

9.4.3 *Penetration Grade Bitumens*

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

9.4.4 *Bitumen Emulsion*

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

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

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

- The rate of application of chippings must be correct so that there is a minimum of excess chippings.
- The second application of binder must be applied before traffic is allowed onto the dressing.
- For a single seal it will be necessary to apply grit or sand after the second application of binder.

9.4.5 *Cutback Bitumens*

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

In Ethiopia, the range of binders available to the engineer may be restricted. In this situation it may then be necessary to blend two grades together or to ‘cut-back’ a supplied grade with diesel oil or kerosene in order to obtain a binder with the required viscosity characteristics. Diesel oil, which is less volatile than kerosene and is generally more easily available, is preferable to kerosene for blending purposes. Only relatively small amounts of diesel oil or kerosene are required to modify a penetration grade bitumen such that its viscosity is suitable for surface treatment at road temperatures in Ethiopia. For example, Figure 9-3 shows that between 2 and 10 per cent of diesel oil was required to modify 80/100 pen bitumen to produce binders with viscosities within the range of road temperatures of between 40°– 60°, which prevail in Ethiopia (Figure 9-2). Figure 9-4 shows the temperature/viscosity relationships for five of the blends made for trials in Kenya.

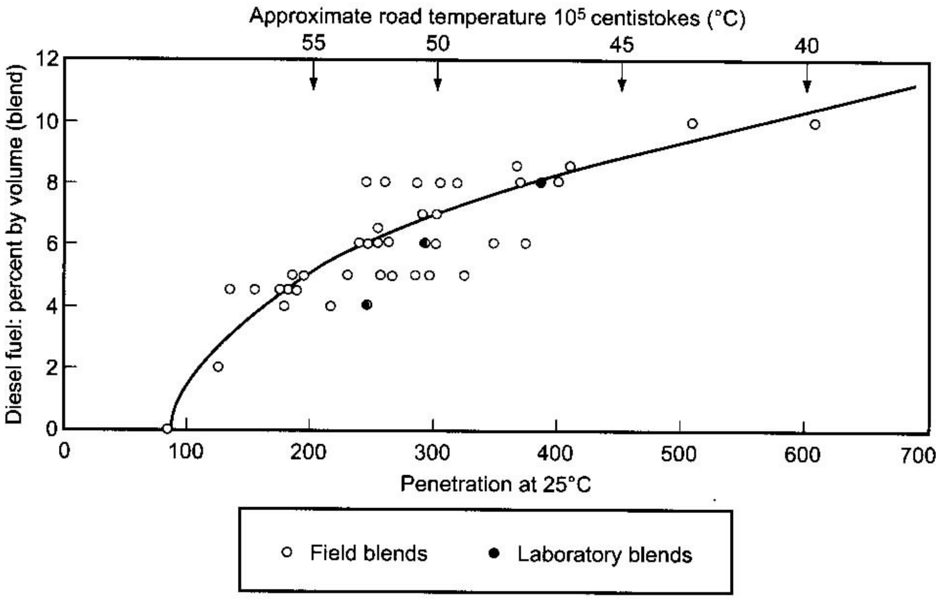


Figure 9-3: Blending Characteristics of 80/100 Pen Bitumen with Diesel Fuel

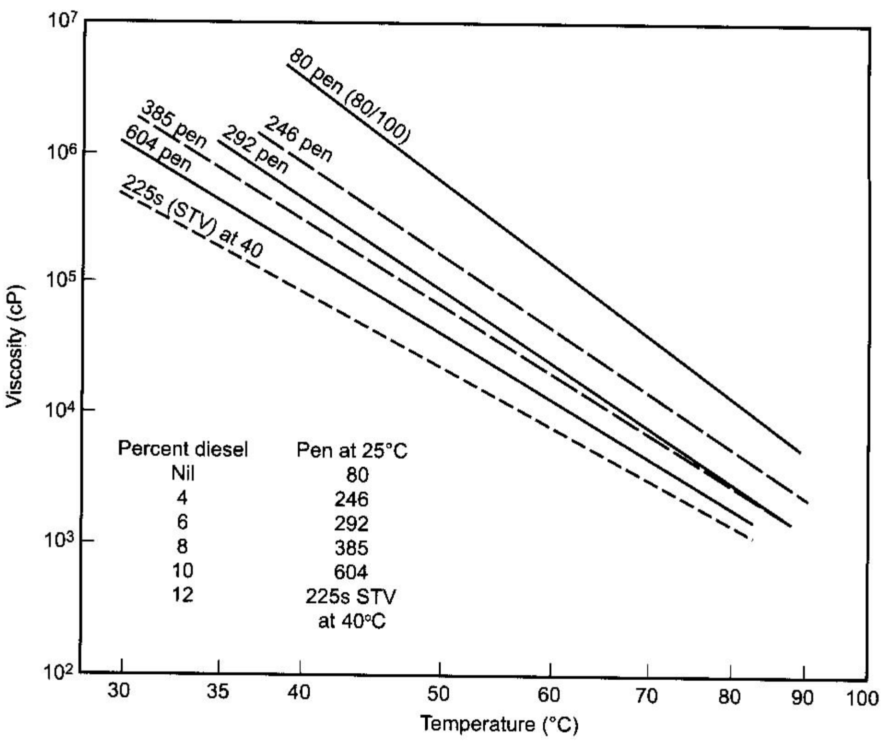


Figure 9-4: Viscosity/Temperature Relationahips for Blends of 80/100 Pen Bitumen with Diesel Fuel

9.4.6 Polymer Modified Bitumens

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

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

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

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

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

9.4.7 Adhesion Agents

Fresh hydrated lime can be used to enhance adhesion. It can be mixed with the binder in the distributor before spraying (slotted jets are probably best suited for this) or the chippings can be pre-coated with the lime just before use, by spraying with lime slurry. The amount of lime to be blended with the bitumen should be determined in laboratory trials but approximately 12 per cent by mass of the bitumen will improve bitumen-aggregate adhesion and it should also improve the resistance of the bitumen to oxidative hardening (Ref. 18).

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

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

9.5 Design

The key stages in the surface treatment design procedure are illustrated in Figure 9-5.

9.5.1 Existing Site Conditions

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

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

Guidance on the selection of chipping size for single surface treatments, relating the nominal size of chipping to the hardness of the underlying road surface and the weight of traffic expressed in terms of the number of commercial vehicles carried per lane per day, is shown in Table 9-6.

Road surface hardness may be assessed by a simple penetration probe test (Ref. 19). This test utilizes a modified soil assessment cone penetrometer and is described briefly in Appendix F. Alternatively the hardness of the existing road surface may be made on the basis of judgement with the help of the definitions given in Table 9-7 (see also Appendix A).

If larger sized chippings are used than those recommended in Table 9-6 then the necessary bitumen spray rate, required to hold the chippings in place, is likely to be underestimated by the design procedure described in this subchapter. This is likely to result in the ‘whip-off’ of chippings by traffic early in the life of the dressing and also to have a significant effect on the long term durability of low volume roads.

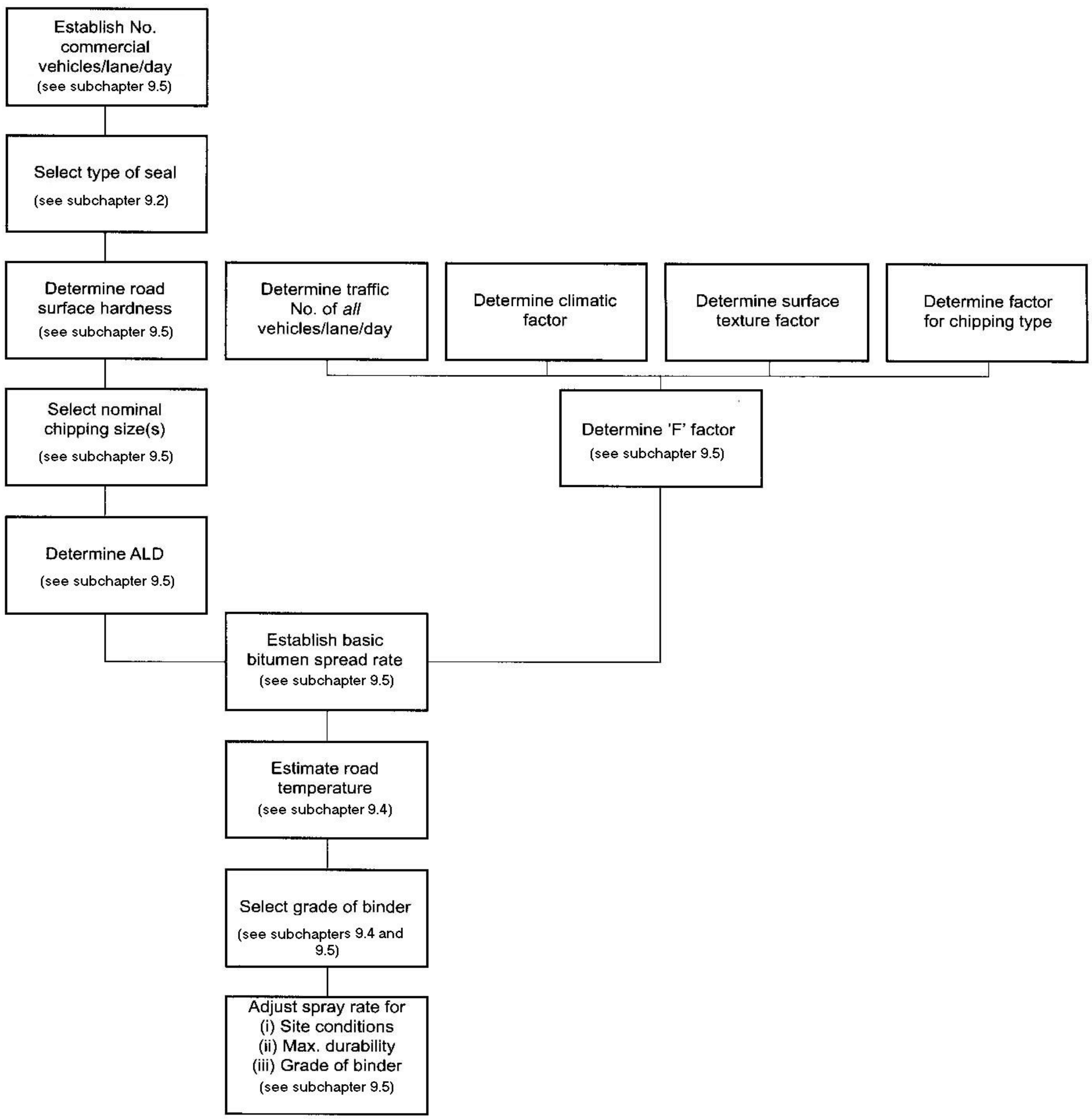


Figure 9-5: Outline Procedure for Design of Surface Treatments

Table 9-6: Recommended Nominal Size of Chippings (mm)

	Approximate Number of Commercial Vehicles with an Unladen Weight Greater Than 1.5 Tonnes Currently Carried per Day in the Design Lane				
Type of Surface	***2000-4000***	***1000-2000***	***200-1000***	***20-200***	***Less Than 20***
Very Hard	10	10	6	6	6
Hard	14	14	10	6	6
Normal	20[1]	14	10	10	6
Soft	*	20[1]	14	14	10
Very Soft	*	*	20[1]	14	10

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

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

**** Unsuitable for surface treatment.***

Table 9-7: Categories of Road Surface Hardness

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

1 ***See Appendices A and F***

In selecting the nominal size of chippings for double surface treatments the size of chipping for the first layer should be selected on the basis of the hardness of the existing surface and the traffic category as indicated in Table 9-6. The nominal size of chipping selected for the second layer should preferably have an ALD of not more than half that of the chippings used in the first layer. This will promote good interlock between the layers.

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

9.5.2 Selecting the Binder

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

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

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

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

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

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

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

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

9.5.3 Choice of Binder and Timing of Construction Work

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

MC 3000 cutback binder typically contains 12 to 17 per cent of cutter. Under warm road conditions this makes the binder very tolerant of short delays in the application of chippings and of the use of moderately dusty chippings. It is therefore a good material to use. However, a substantial percentage of the cutter, especially if it is diesel, can remain in the seal for many months. If road temperatures increase soon after construction, it is likely that MC 3000 will be found to be 'tender' and that the seal can be easily damaged. This should not be a problem for lightly trafficked roads and for new roads that are not opened to general traffic for several days after the surface treatment is constructed. If a road must be opened to fast high volume traffic within a few hours of construction then there will be considerable advantage in using as high a viscosity binder as conditions will permit. For instance, if the road temperature is 40^{o}C ten for heavy traffic the chart in Figure 9-2 would suggest that MC 3000 would be only just viscous enough. 400/500 penetration grade bitumen would be on the limit of being too viscous, however, it would be preferable to cut-back the bitumen to a 500/600 penetration grade rather than use a MC 3000 grade. If pre-coated chippings could be used then the use of a 400 penetration grade bitumen would be acceptable.

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

In either situation, early trafficking is very likely to dislodge chippings and seriously damage the seal.

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

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

9.5.4 Designing the Surface Treatment

Having selected the nominal size of chipping and the type of binder to be used, the next step in the design of a surface treatment is to determine the rate of spread of the binder. Differences in climate, uniformity of road surfaces, the quality of aggregates, traffic characteristics and construction practice, necessitate a general approach to the determination of the rate of spread of the binder for application in Ethiopia.

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

9.5.5 Determining the Average Least Dimension (ALD) of Chippings

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

Method A: A grading analysis is performed on a representative sample of the chippings in accordance with ASTM C136. The sieve size through which 50 per cent of the chippings pass is determined (i.e. the 'median size'). The flakiness index is then also then derived from the nomograph shown in Figure 9-6.

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

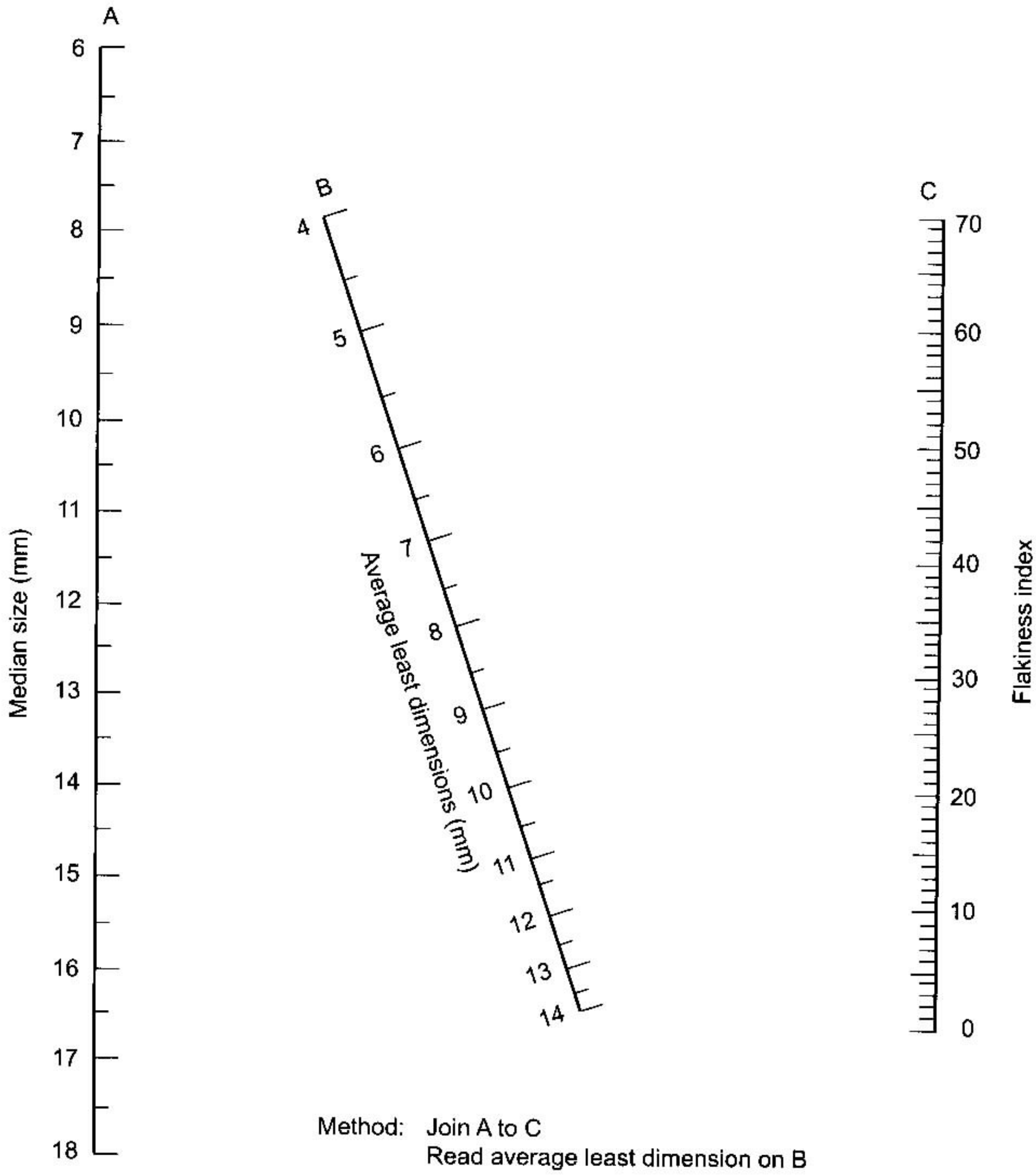


Figure 9-6: Determination of Average Least Dimension

9.5.6 Determining the Overall Weighting Factor

The ALD of the chippings is used with an overall weighting factor to determine the basic rate of spray of bitumen. The overall weighting factor ‘F’ is determined by adding together four factors that represent; the level of traffic, the condition of the existing road surface, the climate and the type of chippings that will be used. Factors appropriate to the site to be surface dressed are selected from Table 9-8.

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

-2 –1 +6 +1 = +4

Table 9-8: Weighting Factors for Surface Treatment Design

Description		Factor
Total traffic (all classes)	***Vehicles/lane/day***	
Very light	0 – 50	+3
Light	50 – 250	+1
Medium	250 – 500	0
Medium-Heavy	500 – 1500	-1
Heavy	1500 – 3000	-3
Very Heavy	3000+	-5
Existing Surface		
Untreated or primed base		+6
Very lean bituminous		+4
Lean bituminous		0
Average bituminous		-1
Very rich bituminous		-3
Climatic Conditions		
Wet and cold		+2
Tropical (wet and hot)		+1
Temperate		0
Semi-arid (hot and dry)		-1
Arid (very dry and very hot)		-2
Type of Chippings		
Round/dusty		+2
Cubical		0
Flaky (see Tables 9-1 and 9-2)		-2
Pre-coated		-2

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

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

9.5.7 *Determining the Basic Bitumen Spray Rate*

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

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

Where F = Overall weighting factor
ALD = The average least dimension of the chippings (mm)
R = Basic rate of spread of bitumen (kg/m^2)

Alternatively, the values for F and ALD can be used in the design chart given in Figure 9-7. The intercept between the appropriate factor line and the ALD line is located and the rate of spread of the binder is then read off directly at the bottom of the chart. The basic rate of spread of bitumen (R) is the mass of MC 3000 binder per unit area on the road surface immediately after spraying. The relative density of MC 3000 can be assumed to be 1.0 and the spread rate can therefore also be expressed in 1iters/m^2; however, calibration of a distributor is easier to do by measuring spray rates in terms of mass.

9.5.8 *Spray Rate Adjustment Factors*

Best results will be obtained if the basic rate of spread of binder is adjusted to take account of traffic speed and road gradient as follows:

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

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

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

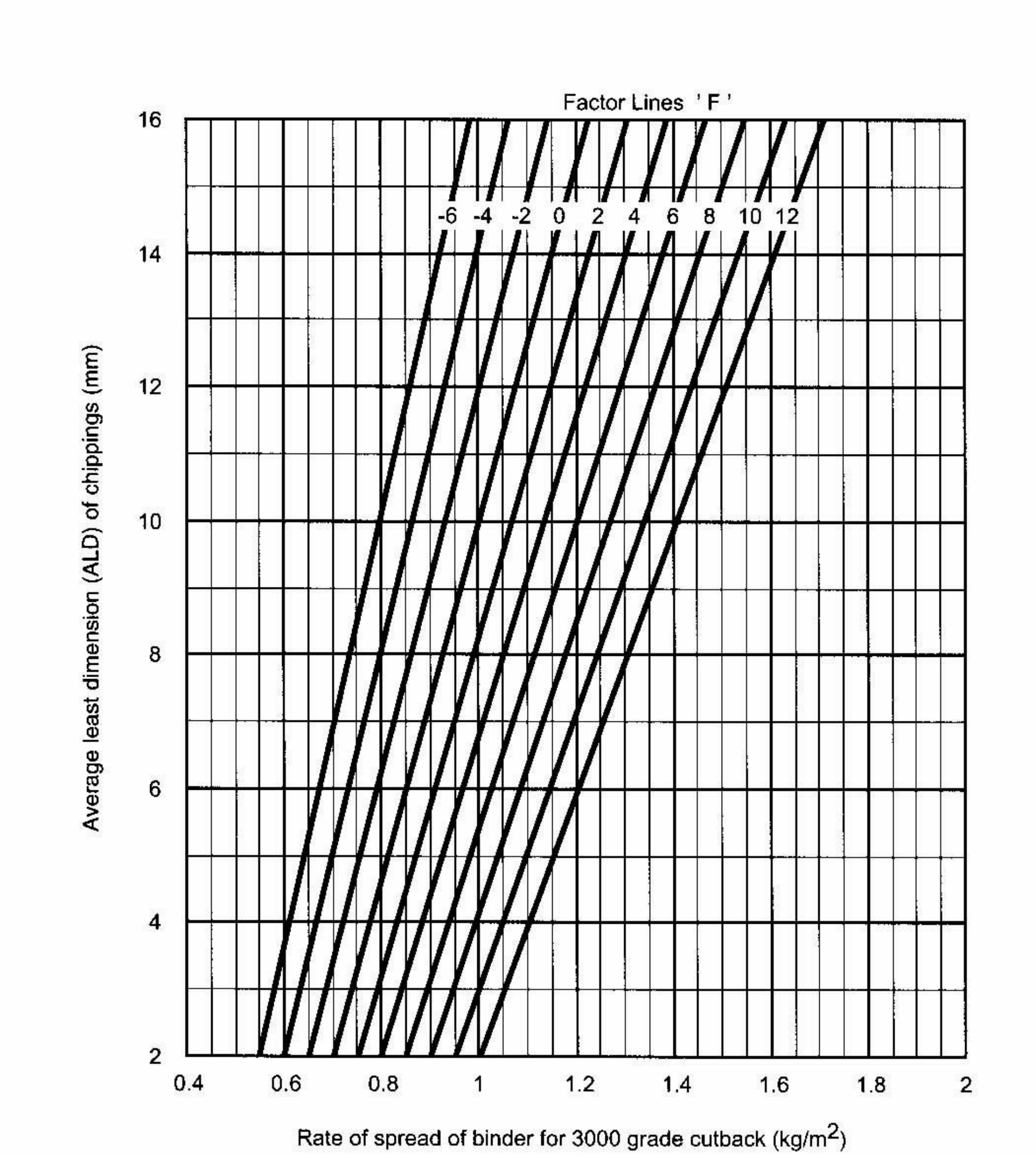


Figure 9-7 Surface Treatment Design Chart

The basic rate of spread of binder must also be modified to allow for the type of binder used. The following modifications are appropriate:

- *Penetration grade binders*: decrease the rate of spread by 10 per cent.
- *Cutback binders*: for MC/RC 3000 no modification is required (in the rare cases when cutbacks with lower viscosity are used the rate of spread should be increased to allow for the additional percentage of cutter used).

Suggested adjustment factors for different binders and different site conditions are given in Table 9-9. The adjustment factors reflect the amount of cutter used in the base 80/100 penetration grade bitumen but must be regarded as approximate values.

Table 9-9: Typical Bitumen Spray Rate Adjustment Factors

Binder Grade	*Basic Spray Rate from Figure 9-7 or Equation 1*	*Flat Terrain, Moderate Traffic Speed*	*High Speed Traffic, Down-Hill Grades>3%*	*Low Speed Traffic, Up-Hill Grades>3%*
MC 3000	R	R	R*1.1	R*0.9
300 pen	R	R*0.95	R*1.05	R*0.86
80/100 pen	R	R*0.9	R*0.99	R*0.81
Emulsion[1]	R	R*(90/% binder)	R*(99/% binder)	R*(81/% binder)

[1] ***'% binder' is the percentage of bitumen in the emulsion.***

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

If a different grade of binder is required then the adjustment factor should reflect the different amount of cutter used. For instance, a 200 penetration binder may have 3 per cent cutter in it and therefore the spray rate is 103 per cent of the rate for a 80/100 penetration bitumen. Subchapter 9.6 gives an example of the use of the design chart and adjustment factors.

9.5.9 Adjusting Rates of Spray for Maximum Durability

The spray rate which will be arrived at after applying the adjustment factors in Table 9-9 will provide very good surface texture and use an 'economic' quantity of binder. However, because of the difficulties experienced in carrying out effective maintenance, there is considerable merit in sacrificing some surface texture for increased durability of the seal. For roads on flat terrain and carrying moderate to high-speed traffic it is possible to increase the spray rates obtained from Table 9-9 by approximately 8 per cent. The heavier spray rate may result in the surface having a 'bitumen-rich' appearance in the wheel paths of roads carrying appreciable volumes of traffic. However, the additional binder should not result in bleeding and it can still be expected that more surface texture will be retained than is usual in an asphalt concrete wearing course.

9.5.10 Surface Treatment Design for Low Volume Roads

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

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

Where crushing facilities are put in place solely to produce chippings for a project, it will be important to maximize use of the crusher output. This will require the use of different combinations of chipping sizes and correspondingly different bitumen spray rates. The normally recommended sizes of chippings for different road hardness and low commercial traffic volumes are given in Table 9-10.

Table 9-10: Nominal Size of Chippings for Different Hardness of Road Surface

No. of Commercial Vehicles/lane/day[1]	***20-100***	***<20***
Category of Road Surface Hardness	***Nominal Chipping Size (mm)***	
Very hard	6	6
Hard	6	6
Normal	10	6
Soft	14	10

[1] ***Vehicles with an unladen weight greater than 1.5 tonnes***

It may be desirable to use chippings of a larger size than those recommended in Table 9-10 for reasons of economy. It is likely that the rate of application of bitumen determined in the normal way will be too low to obtain good durability. Low volumes of traffic are also unlikely to cause the chippings to be ‘rotated’ into a tight matrix and this will result in the layer being of greater depth than the ALD of the chippings, which is assumed in the design process. It should therefore be safe to increase bitumen spray rates on low volume roads to compensate for the reduced embedment of ‘oversize’ chipping and the increased texture depth that results from less re-orientation of the chippings under light traffic.

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

It is suggested that on low volume roads the bitumen spray rates should be increased above the basic rate of spread of bitumen indicated above by up to the percentages given in Table 9-11. It is important that these increased spray rates are adjusted on the basis of trial sections and local experience.

9.5.11 SPREAD RATE OF CHIPPINGS

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

Chipping application rate (kg/m^2) = 1.364*ALD (2)

Table 9-11: Suggested Maximum Increases in Bitumen Spray Rate for Low Volume Roads

ALD of chippings (mm)	3		6		>6	
All traffic (vehicles/lane/day)	<20	20-100	<20	20-100	<20	20-100
Increase in bitumen spray rate (per cent)	15	10	20	15	30	20

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

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

9.6 Example of a Surface Treatment Design

Site Description

A two-lane trunk road at an altitude of approximately 1500m.
Vehicle count averaged 3370 per day/lane (i.e. ‘Heavy’ rating).

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

Design	**Factor**
Traffic (Heavy)	-3
Existing Surface (Average Bituminous)	-1
Chippings (Cubical)	0
Climate (Hot/Dry)	-1
Overall Weighing Factor (F)	-5

Aggregate (Nominal 19 mm)

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

The determination of spread rates of 80/100 and 400 pen bitumen for an F factor of –5 and an ALD of 12 on a site where maximum durability is required are summarized in Table 9-12.

Table 9-12: Determination of Spread Rates for 400 Penetration Grade Bitumen

Type of Terrain	*Basic Spread Rate R for MC 3000 (from Fig. 9-7 or Equation 1) (kg/m_2)*	*For Increased Durability R_D = (R* 1.08) (kg/m_2)*	*Spread Rates for Penetration Grade Binders (kg/m^2)*	
			*80/100 pen (R_D*0.9)*	*400 pen (R_D*0.9*1.067)*
Flat	0.89	0.96	0.87	0.92
Uphill Grade > 3%	0.89*0.9 = 0.80	0.87	0.78	0.84
Downhill Grade > 3%	0.89*1.1 = 0.98	1.06	0.95	1.02

1. For slow traffic or climbing grades steeper than 3 per cent, reduce the rate of spread of binder by 10 per cent.
2. For fast traffic or down grades steeper than 3 per cent increase the rate of spread of binder by 10 to 20 per cent.

9.7 Other Surface Treatments

There are several other kinds of surface treatments, five of which are described briefly below.

9.7.1 Slurry Seals and Cape Seals

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

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

Both anionic and cationic emulsions may be used in slurry seals but cationic emulsion is normally used in slurries containing acidic aggregates, and its’ early breaking characteristics are also advantageous when rainfall is likely to occur. Suitable specifications for slurry seals and for a Cape-seal are given in Tables 9-13 and 9-14.

Table 9-13: Aggregate Particle Size Distribution for Slurry Seals

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

The optimum mix design for the aggregate, filler, water and emulsion mixture should be determined using ASTM D 3910-84 (1996).

Table 9-14: Typical Coverage for a New 'Cape Seal'

Size of Chipping in Surface treatment (mm)	*Coverage (m^2/m^3)*
20	130-170
14	170-240
10	180-250

9.7.2 OTTA SEAL

An Otta seal is different to surface treatment in that a graded gravel or crushed aggregate containing all sizes, including filler, is used instead of single sized-chippings. There is no formal design procedure but recommendations based on case studies have been published (Ref. 20). An Otta seal may be applied in a single or double layer. Evidence on the performance of these types of seal has shown them to carrying up to 300 vehicles per day (Ref. 21).

The grading of the material is based on the level of traffic expected. Recommended grading envelopes are given in Table 9-15. Generally for roads carrying light traffic (<100 vehicles per day), a 'coarse' grading should be chosen while a 'dense' grading should be applied to one carrying greater than 100 vehicles per day.

Table 9-15: Otta Seal Aggregate Grading Requirements

	Percentage Passing[1]	
Sieve (mm)	***Dense***	***Coarse***
19	100	100
16	79-100	77-100
12	61-100	59-100
9.5	42-100	40-85
4.75	19-68	17-46
2.36	8-51	1-20
1.18	6-40	0-10
0.60	3-30	0-3
0.30	2-21	0-2
0.15	1-16	0-1
0.075	0-10	0-1

[1] ***Aggregate should be screened to remove stone greater than 19mm***

The viscosities of binders used in construction should reflect the quality of aggregate employed but normally cut back bitumen MC 800, MC 3000 or 150/200 penetration grade bitumen is used depending upon the traffic volumes and type of aggregate cover. Spray rates cannot be calculated by design and must be chosen empirically. Typically, spray rates (hot) for single seals are between 1.6 and 2.0 $1/m^2$ so that necessary detailed adjustments can be made.

It is because of the broad range of materials that may be used and the empirical nature of the design of this type of seal that it is imperative that pre-construction trials be carried out. This strategy will identify any special local conditions concerning the available aggregates and binders to become apparent to enable the engineer to adjust the nominal design.

An important aspect of Otta seal construction is the need for extensive rolling by pneumatic rollers for two or tree days after construction. The action of rolling ensures the binder is forced upwards, coating the aggregate, and thereby initiating the process, continued by subsequent trafficking, of forming a premix like appearance to the surface.

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

9.7.3SAND SEALS

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

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

The sand should be a clean coarse sand, with a maximum size of 6mm, containing no more than 15 per cent of material finer than 0.3mm and a maximum of 2 per cent of material finer than 0.15mm. The sand should be applied at a rate of 6 to 7×10^{-3} m^3/m^2 (Ref. 22). The binder, which may be a cutback or an emulsion, should be spread at a rate of approximately 1.0 to 1.2 kg/m^2 depending on the type of surface being sealed.

9.7.4 Synthetic Aggregate and Resin Treatments

These treatments are costly and are used only on relatively small areas usually in urban situations, where high skidding resistance is required. The aggregate is normally a small single-sized, calcined bauxite which has a high resistance to polishing under traffic. The aggregate is held by a film of epoxy-resin binder (Ref. 23). The process requires special mixing and laying equipment and is normally undertaken by specialist contractors.

9.7.5 Applications of Light Bitumen Sprays

There are two main uses for light sprays of bitumen:

- A light film of binder which can be applied as the final spray on a new surface treatment. The advantage of this procedure is that the risk of whip-off of chippings under fast traffic is reduced. This is particularly useful where management of traffic speed is difficult.

- A light spray of binder can be used to extend the life of a bituminous surfacing. This is particularly useful where a surfacing is showing signs of bitumen aging by fretting or cracking.

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

Fog Sprays. A light spray of bitumen emulsion is ideal for improving early retention of chippings in a new dressing (Ref. 22). The road surface is usually dampened before spraying or, if a low bitumen content emulsion (45 per cent) is available, this dampening can be omitted. Complete breaking of the emulsion must occur before traffic is allowed onto the dressing and it may be necessary to dust the surface with sand or crusher fines to prevent pick-up by traffic. If emulsion is diluted with water, to obtain a 45 per cent bitumen content to ensure the bitumen will flow around the chippings, then the suitability of the water must be established by mixing small trial batches.

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

Enrichment Sprays. Surfaces which are showing obvious signs of disintegration through bitumen aging can be enriched by applying stable grade anionic bitumen emulsion which has been diluted at a rate of 1:1 with water (Ref. 22). The rate of application will depend upon the texture of the surfacing and this must be determined by trial sprays, however, it is likely to be between 0.2 and 0.5 $liters/m^2$ of residual bitumen. Great care must be taken to avoid leaving a slippery surface and a light application of sand sized fines may be required in some cases.

10. FLEXIBLE PAVEMENT DESIGN CATALOG

10.1 Description of the Catalog

The design of flexible pavements, as given in this manual, is based on the catalog of pavement structures of TRL Road Note 31 (Ref. 1).

Before the catalog is used, the elements described in Chapters 2 and 3 regarding traffic and subgrade should be considered. Simultaneously, the information regarding availability, costs and past experience with materials should be gathered.

The catalog offers, in eight different charts, alternative pavement structures for combinations of traffic and subgrade classes. The various charts correspond to distinct combinations of surfacing and roadbase materials, as shown in Table 10-1:

Table 10-1: Summary of Material Requirements for the Design Charts

CHART NO	SURFACING	BASE COURSE	REFER TO CHAPTERS
1	Double surface dressing	T1 -T4 use GB1,GB2 or GB3 T5 use GB1 or GB2 T6 must be GB1	6 and 9
2	Double surface dressing	T1 -T4 use GB1,GB2 or GB3 T5 use GB1 or GB2 T6 – T8 must be GB1	6, 7 and 8
3	“Flexible” asphalt	T1 –T5 use GB1 or GB2 T6 use GB1	6 and 8
4	“Flexible” asphalt	T1 –T5 use GB1 or GB2 T6 – T8 use GB1	6, 7 and 8
5	Wearing course and Base course	GB1	6 and 8
6	Wearing course and Base course	GB1 or GB2	6, 7 and 8
7	High quality single seal or double seal for T4 “Flexible” asphalt for T5-T8	RB1, RB2 or RB3	8 and 9
8	Double surface dressing	CB1, CB2	7 and 9

All the charts provide alternate pavement structures for all subgrade classes (S1 through S6). They are not however suitable for all classes of traffic, as some structures would be neither technically appropriate nor economically justified.

10.2 Use of the Catalog

Although the thicknesses of layers should follow the design charts whenever possible, some limited substitution of materials between subbase and selected fill is allowable based on the structural number principles outlined in the AASHTO Guide for Design of Pavement Structures (Ref. 6). Where substitution is allowed, a note is included with the design chart.

In Charts 3, 4 and 7 where a thin surfacing of asphalt concrete is defined, it is important that the surfacing material should be able to withstand some deformation and that the granular roadbase (bitumen stabilized in the case of Chart 7) be of the highest quality crushed stone. This latter point is particularly important for the higher classes of traffic (classes T5 through T8). For the asphalt concrete, the mix design should favor durability over seeking a high stability.

The above requirement for high quality roadbase also applies to classes of traffic T5 and higher in Charts 1 and 2 using surface treatment as surfacing, but a gravel roadbase may be considered for the lower classes (T1-T4). The same requirement always applies to the granular roadbase of Chart 5 and to the granular roadbase component of the composite roadbase of Chart 6.

For lime or cement-stabilized materials (Charts 2, 4, 6 and 8), the charts define the layers with different symbols and thereby indicate the underlying assumptions regarding the strength of material.

The choice of chart will depend on a variety of factors but should be based on minimizing total transport costs. Factors that will need to be taken into account in a full evaluation include:

- the likely level and timing of maintenance
- the probable behavior of the structure
- the experience and skill of the contractors and the availability of suitable equipment
- the cost of the different materials that might be used
- other risk factors

It is not possible to give detailed guidance on these issues. The charts have been developed on the basis of reasonable assumptions concerning the first three of these and therefore the initial choice should be based on the local costs of the feasible options. If any information is available concerning the likely behavior of the structures under the local conditions, then a simple risk analysis can also be carried out to select the most appropriate structure. For many roads, especially those that are more lightly trafficked, local experience will dictate the most appropriate structures and sophisticated analysis will not be warranted.

10.3 Design Example

An example of traffic calculations was given in Chapter 2 for a particular section of a trunk road. In the example, a traffic class T8 has been derived (with a total of ESAs on the order of 20 millions over the design period).

From Table 10-1 given above, for that class of traffic, it is readily apparent that the use of the design charts in the catalog of structures is narrowed down to Charts 4 through 7. From the same table, without further information regarding the subgrade and the materials, it would also appear that any type of surfacing is possible, as well as several types of roadbase.

The subgrade strength has reasonably been ascertained (cf. Section 3.2) to be represented by CBRs in the range of 5 to 7, considering that some portions of the alignment which might exhibit higher strength are so limited in number and extent that it makes it impractical to consider several designs. The subgrade strength class to be assigned to this project is therefore S3 (cf. Table 3-1 and Figure 10-1).

The following preliminary information has been derived from the investigations and simple cost comparison:

- The materials which may be considered for cement- or lime-stabilization have relatively low percentages of fines and low plasticity, thus making cement-stabilization more promising.
- Granular subbase materials are available in sufficient quantities and cement stabilization of the subbase is uneconomical when compared to bank-run materials. Stabilization of subbase materials will not be further considered.
- All other materials entering the composition of the possible pavement structures are available, albeit in various quantities and associated transport/construction costs.

Based on the above, and with the T8/S3 combination of traffic and subgrade strength classes, the design charts 4 through 7 indicate the possible alternate pavement structures given in Table 10-2.

Table 10-2: Design Example: Possible Pavement Structures

	Design Chart No.	4	5	6	7
Pavement Components and Selected Fill	Possible Alternate Pavement Structures	Alternate Structure No. 1	Alternate Structure No. 2	Alternate Structure No. 3	Alternate Structure No. 4
Surfacing (asphalt concrete) (1)		5 cm AC	15cm AC	15cm AC	5 cm AC
Roadbase:					
· Crushed Stone		15 cm	25 cm	15 cm	—
· Cement stabilized (e.g. 4 Mpa)		15 cm	—	—	—
· Cement stabilized (e.g. 2.5 Mpa)		12.5 cm	—	22.5 cm	—
· Bituminous stabilized		—	—	—	20 cm
Granular subbase		—	27.5 cm	—	27.5 cm (2)
Selected fill		15 cm	—	—	— (2)

Notes: (1) Asphalt concrete (AC) only alternative for T8
(2) In the alternate structure No. 4, 27.5 cm of granular subbase can be used (Alt. 4a). Alternatively, up to 7.5 cm of granular subbase may be substituted with 10 cm of selected fill (Alt. 4b).

Further analyses of recent contracts, production costs hauling distances and associated costs have established relative costs for the various alternate pavement layers (all costs per m^2 and expressed as a ratio to the highest cost element) as shown in Table 10-3.

With these elements, the relative costs of the possible alternate pavement structures are evaluated as follows in Table 10-4.

Based on the above, the alternate structures including cement stabilized layers (Nos. 1 and 3) appear prohibitive, and the alternate (No. 2) including only crushed stone roadbase and subbase also appear at a disadvantage. The preferred solutions (Nos. 4a and 4b) are only marginally different. It may be advisable to present both alternatives for bidding purposes.

Table 10-3: Design Example: Relative Unit Costs of Materials

MATERIAL		RELATIVE UNIT COST
Asphalt Concrete	5 cm thick	0.33
	15 cm thick*	0.87
Bituminous stabilized roadbase	20 cm thick	1.00
Crushed stone roadbase	15 cm thick	0.56
	25 cm thick	0.90
Cement stabilized roadbase, 4 MPa	15 cm thick	0.81
Cement stabilized roadbase, 2.5 MPa	12.5 cm thick	0.73
	22.5 cm thick	0.91
Granular subbase	20 cm thick	0.29
	15 cm thick	0.39
Select fill	10 cm thick	0.13
	15 cm thick	0.19

*wearing course and binder course

Table 10-4: Relative Costs of the Possible Alternate Pavement Structures

Alternate Pavement Structure No.	Description	Relative Unit Cost
1	5 cm AC +15 cm crushed stone roadbase +15 cm cement stabilized roadbase (4 MPa) +12.5 cm cement stabilized roadbase (2.5 MPa) +15 cm selected fill	2.62
2	15 cm AC +25 cm crushed stone roadbase +27.5 cm granular subbase	2.16
3	15 cm AC +25 cm crushed stone roadbase +22.5 cm cement stabilized roadbase (2.5 Mpa)	2.33
4a	5 cm AC +20 cm bituminous stabilized roadbase +27.5 cm granular subbase	1.73
4b	5 cm AC +20 cm bituminous stabilized roadbase +20 cm granular subbase +10 cm selected fill	1.75

KEY TO STRUCTURAL CATALOGUE

Traffic classes
(10^6 esa)

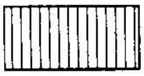
T1 = < 0.3
T2 = 0.3 - 0.7
T3 = 0.7 - 1.5
T4 = 1.5 - 3.0
T5 = 3.0 - 6.0
T6 = 6.0 - 10
T7 = 10 - 17
T8 = 17 - 30

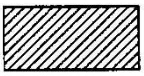
Subgrade strength classes
(CBR%)

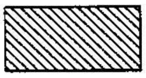
S1 = 2
S2 = 3 , 4
S3 = 5 - 7
S4 = 8 - 14
S5 = 15 - 29
S6 = 30+

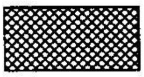
Material Definitions

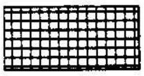

Double surface dressing


Flexible bituminous surface


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

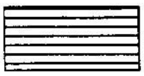

Bituminous roadbase, RB


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

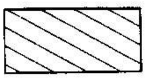

Cement or lime-stabilised sub-base, CS

Figure 10-1: Key to Structural Catalog

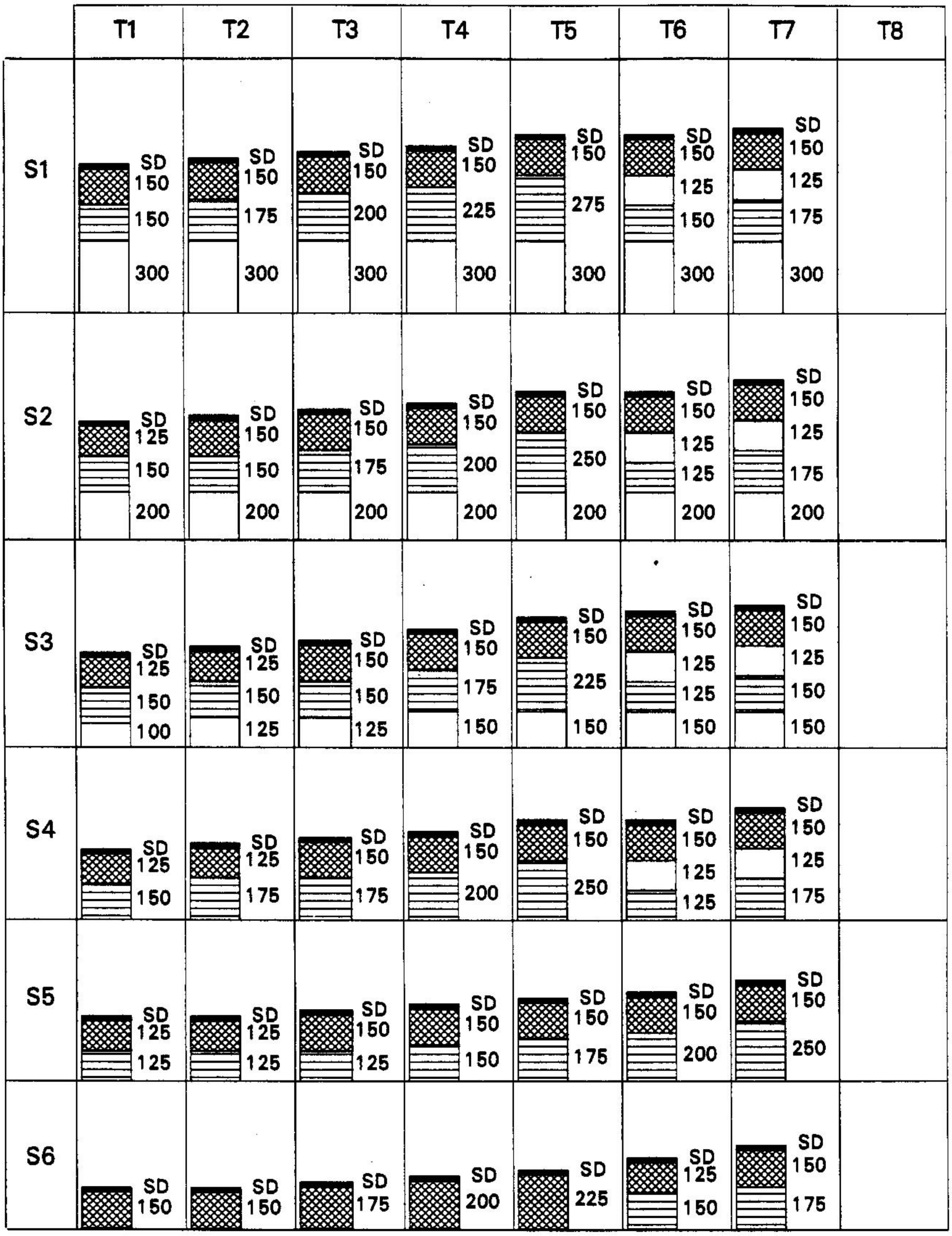
CHART 1 GRANULAR ROADBASE / SURFACE DRESSING

	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*	SD 200 325*	SD 225 350*		
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*	50 175 325*	50 200 350*		
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

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

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.

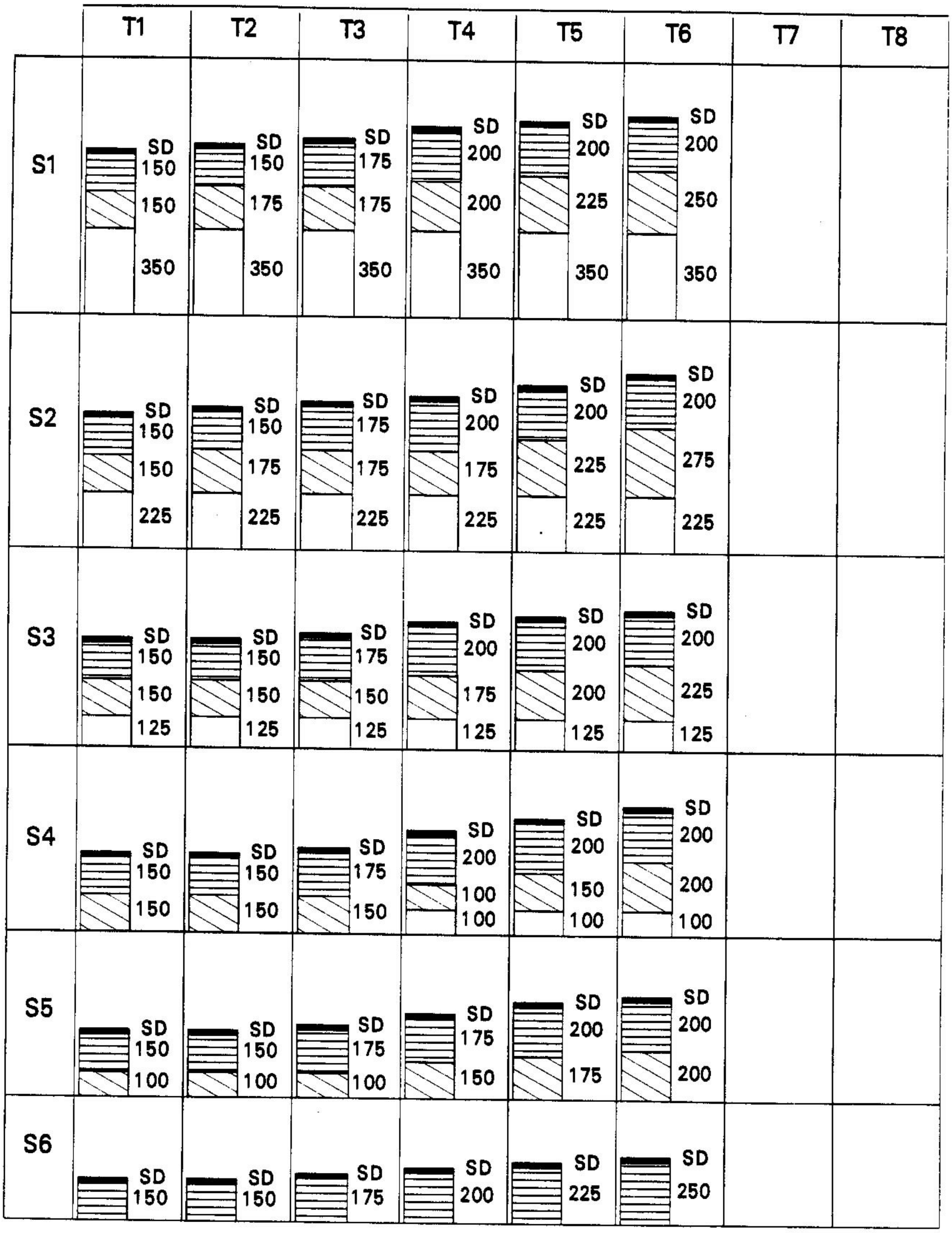
CHART 7 BITUMINOUS ROADBASE / SEMI-STRUCTURAL SURFACE

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

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



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

11. DESIGN OF GRAVEL AND LOW STANDARD ROADS

11.1 General

Much of the information presented in this Section of the Pavement Design Manual is based on the "Pavement and Materials Design Manual" prepared by the United Republic of Tanzania Ministry of Works 1999, and on relevant ERA and TRL publications. Available information has been modified to provide a simple procedure to design gravel wearing courses and low standard roads, which is appropriate to Ethiopian conditions.

Gravel road pavements are generally utilized for roads where design traffic flow Annual Average Daily Traffic (AADT) is less than 200. This Section sets out the standards for pavement design, and specifies the materials which may be used for gravel roads.

11.2 Design Principles

11.2.1 Steps to be considered in the Design Process

1. Traffic (Baseline flow and forecast)
2. Material and geotechnical information (Field survey and material properties)
3. Subgrade (Classification, foundation for expansive soils and material strength)
4. Thickness design (Gravel wearing coarse thickness)
5. Materials design

11.2.2 All-weather Access

An essential consideration in the design of gravel roads is to ensure all-weather access. This requirement places particular emphasis on the need for sufficient bearing capacity of the pavement structure and provision of drainage and sufficient earthworks in flood or problem soil areas (e.g. black cotton).

11.2.3 Surface Performance

The performance of the gravel surface mainly depends on material quality, the location of the road, and the volume of traffic using the road. Gravel roads passing through populated areas in particular require materials that do not generate excessive dust in dry weather. Steep gradients place particular demands on gravel wearing course materials, which must not become slippery in wet weather or erode easily. Consideration should therefore be given to the type of gravel wearing course material to be used in particular locations such as towns or steep sections. Gravel loss rates of about 25-30mm thickness a year per 100 vehicles per day is expected, depending on rainfall and materials properties (particularly plasticity).

Performance characteristics that will assist in identifying suitable material are shown in Figure 11-1.

11.2.4 Maintenance

The material requirements for the gravel wearing course include provision of a gravel surface that is effectively maintainable. Adherence to the limits on oversize particles in the material is of particular importance in this regard and will normally necessitate the use of crushing or screening equipment during material production activities.

11.3 Design Method

The required gravel thickness shall be determined as follows:

1. Determine the minimum thickness necessary to avoid excessive compressive strain in the subgrade (D_1).
2. Determine the extra thickness needed to compensate for the gravel loss under traffic during the period between regravelling operations (D_2).
3. Determine the total gravel thickness required by adding the above two thicknesses (D_1+ D_2).

11.3.1 Minimum Thickness Required

It is necessary to limit the compressive strain in the subgrade to prevent excessive permanent deformation at the surface of the road .Figure 3 gives the minimum gravel thickness required for each traffic category with the required thickness of improved subgrade materials for upper and lower subgrade layers.

11.3.2 Gravel Loss

According to TRL Laboratory Report 673, an estimate of the annual gravel loss is given by the following equation:

$$GL = f\,T^2 / (T^2 + 50)\ (4.2 + 0.092\,T + 3.50\,R^2 + 1.88V)$$

Where

GL = the annual gravel loss measured in mm

T = the total traffic volume in the first year in both directions, measured in thousands of vehicles

R = the average annual rainfall measured in m

V = the total (rise + fall) as a percentage of the length of the road

f = 0.94 to 1.29 for lateritic gravels

= 1.1 to 1.51 for quartizitic gravels

= 0.7 to 0.96 for volcanic gravels (weathered lava or tuff)

= 1.5 for coral gravels

= 1.38 for sandstone gravels

11.3.3 Total Thickness Required

The wearing course of a new gravel road shall have a thickness D calculated from:

$$D = D_1 + N.\ GL$$

Where D_1 is the minimum thickness from Figure 11.3

N is the period between regravelling operations in years

GL is the annual gravel loss

Regravelling operations should be programmed to ensure that the actual gravel thickness never falls below the minimum thickness D_1.

11.4 Pavement and Materials

Depending on the CBR_{design} of the subgrade, improved subgrade layers shall be constructed as required, on which the gravel wearing course is placed.

11.5 Crossfall and Drainage

The crossfall of carriageway and shoulders for gravel roads shall be " 4%" as indicated in ERA's *Geometric Design Manual - 2002*. This is to ensure that potholes do not develop by rapidly removing surface water and to ensure that excessive crossfall does not cause erosion of the surface. Provision of drainage is extremely important for the performance of gravel roads.

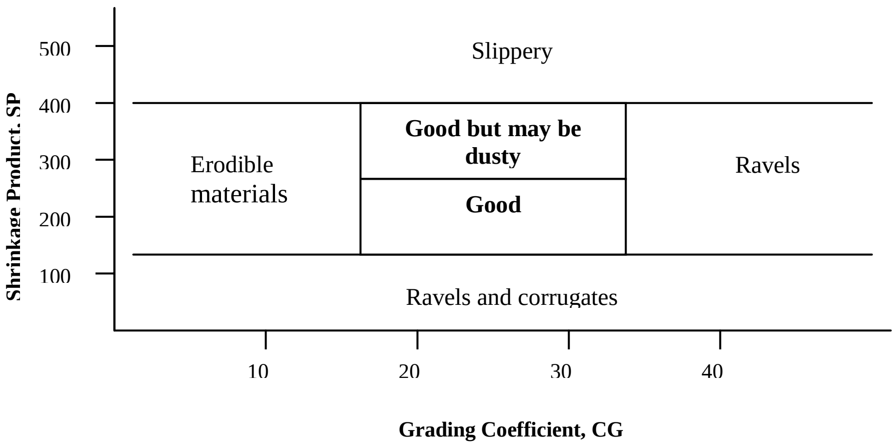
11.6 Material Requirements

11.6.1 Experience with Local Materials

Knowledge of past performance of locally occurring materials for gravel roads is essential. Material standards may be altered to take advantage of available gravel sources provided they have proved to give satisfactory performance under similar conditions.

11.6.2 Marginal Materials

Figure 11-1 illustrates the performance characteristics to be expected of materials that do not meet the requirements for gravel wearing course. Refinements and amendments of the standard material specification may be necessary to overcome problem areas such as towns (dust nuisance) or steep hills (slipperiness).



Note: SP = (Linear Shrinkage) x (% passing 0.425 mm)
GC = ((% passing 28 mm) - (% passing 2 mm)x (% passing 5 mm)/100

Figure 11-1. Expected Performance of Gravel Wearing Course Materials

11.6.3 Improved Subgrade Layer

In General the use of improved subgrade layers has the following advantages:

- Provision of extra protection under heavy axle loads;
- Protection of underlying earthworks;
- Provides running surface for construction traffic;
- Assists compaction of upper pavement layers;
- Provides homogenous subgrade strength;
- Acts as a drainage filter layer;
- More economical use of available materials.

11.6.4 Subgrade CBR

All subgrade materials shall be brought to a strength of at least a minimum CBR of 7% for minor gravel roads and at least a minimum CBR 25 % for major gravel roads. The different types of gravel roads are explained in Section 11.7.

11.6.5 Treatment of Expansive Formations

The following treatment operations should be applied on Expansive Formations for higher class roads of $AADT_{design}$ greater than 50:

i) Removal of Expansive Soil

   a) Where the finished road level is designed to be less than 2 metres above ground level, remove the expansive soil to a minimum depth of 600 mm over the full width of the road, or

   b) Where the finished road level is designed to be greater than 2 metres above ground level, remove the expansive soil to a depth of 600 mm below the ground level under the unsurfaced area of the road structure, or

   c) Where the expansive soil does not exceed 1 meter in depth, remove it to its full depth.

ii) Stockpile the excavated material on either side of the excavation for subsequent spreading on the fill slopes so as to produce as flat a slope as possible.

iii) The excavation formed as directed in paragraph (i) should be backfilled with a plastic non-expansive soil of CBR value 3 - 4 or better, and compacted to a density of 95% modified AASHTO.

iv) After the excavated material has been replaced with non-expansive material in 150mm lifts to 95% modified AASHTO density, bring the road to finished level in approved materials, with a side slope of 1:2, and ensure that pavement criteria are complied with; the previously stockpiled expansive soil excavated as directed under (i) should then be spread over the slope.

v) Do not construct side drains unless they are absolutely essential to stop ponding; where side drains are necessary, they should be as shallow as possible and located as far from the toe of the fill as possible.

vi) Ideally, construction over expansive soil should be done when the in-situ moisture content is at its highest, i.e. at the end of rainy season.

The following treatment operations may be applied on Expansive Formations for light traffic class roads of $AADT_{design}$ less than 50:

i) Remove 150mm of expansive topsoil and stockpile conveniently for subsequent use on shoulder slopes

ii) Shape road bed and compact to 90% modified AASHTO

iii) The excavation formed as directed in paragraph (i) should be backfilled with a plastic non-expansive soil of CBR value 3 - 4 or better, and compacted to a density of 95% modified AASHTO in each 150mm layer; the subgrade material may be plastic but non-expansive.

11.6.6 *Material Characteristics*

Soils used for improved subgrade layers shall be non-expansive, non-dispersive and free from any deleterious matter. They shall comply with the requirements shown in Table 11-1.

Table 11-1

Material Properties	**G20 (Upper Layer)**	**G7 (Lower Layer)**
CBR Dry Climatic Zones (See Note)	Minimum 20 after 4 days soaking	Minimum 7 after 4 days soaking
CBR Wet Climatic Zones (See Note)	Minimum 20 at OMC Minimum 7 after 4 days soaking	Minimum 7 at OMC Minimum 3 after 4 days soaking
PI [%]	Maximum 25	Maximum 30
Compacted Density	95% of AASHTO T180	95% of AASHTO T180
Maximum particle size	2/3 of layer thickness	2/3 of layer thickness
Compacted layer thickness	Maximum 200 mm	Maximum 250 mm
Note: Climatic Zones for Ethiopia are described in Section 11.10.		

11.7 Gravel Wearing Course

11.7.1 *Performance Characteristics of Gravel Wearing Course*

The materials for gravel wearing course should satisfy the following requirements that are often somewhat conflicting:

a) They should have sufficient cohesion to prevent ravelling and corrugating (especially in dry conditions)

b) The amount of fines (particularly plastic fines) should be limited to avoid a slippery surface under wet conditions.

Figure 11-1 shows the effect of the Shrinkage Product (SP) and Grading Coefficient (GC) on the expected performance of gravel wearing course materials. Excessive oversize material in the gravel wearing course affects the riding quality in service and makes effective shaping of the surface difficult at the time of maintenance. For this reason the following two types of gravel wearing course material are recommended. Type 1 gravel wearing course which is one of the best material alternatives which shall be used on all roads which have $AADT_{design}$ greater than 50. Type 1 material shall also be used for all routine and periodic maintenance activities for both major and minor gravel roads. Type 1 or Type 4 gravel wearing course material may be used on new construction of roads having $AADT_{design}$ less than 50. Other alternatives are also specified in this chapter.

11.7.2 Gravel Wearing Course Material Specification

Selected material shall consist of hard durable angular particles of fragments of stone or gravel. The material shall be free from vegetable matter and lumps or balls of clay.

Type 1

The grading of the gravel after placing and compaction shall be a smooth curve within and approximately parallel to the envelopes detailed in Table 11-2.

The material shall have a percentage of wear of not more than 50 at 500 revolutions, as determined by AASHTO T96.

The material shall be compacted to a minimum in-situ density of 95% of the maximum dry density determined in accordance with the requirements of AASHTO T 180.

The plasticity index should be not greater than 15 and not less than 8 for wet climatic zones and should be not greater than 20 and not less than 10 for dry climatic zones.

The linear Shrinkage should be in a range of 3-10%.

Note that the above gradation and plasticity requirements are only to be used with angular particles and that crushing and screening are likely to be required in many instances for this purpose.

Type 2 & 3

These materials may be more rounded particles fulfilling the following:

a) The Plasticity Index lies in a range of 5-12% in wet areas, and in any case less than 16% in other areas

b) The materials have the sanction of local experience

Use of more rounded particles may allow the use of river gravel. Trials should nevertheless be conducted to verify whether crushing occurs under traffic or whether crushing should be considered prior to use. Subject to trials, a minimum percentage by weight of particles with at least one fractured face of 40% may be considered. This requirement may also be expressed in terms of crushing ratio.

Except for very low traffic (less than 15 vehicles per day), the CBR should be in excess of 20 after 4 days of soaking at 95% of maximum dry density under Heavy Compaction. For very low traffic, the requirement may be relaxed to a CBR of 15.

Type 4

This material gradation allows for larger size material and corresponds to the gradation of a base course material. The use of this gradation of materials is subject to the local experience and shall be used with PIs in a range of 10-20.

Type 5 & 6

These materials gradations are recommended for smaller size particles. They may be used if sanctioned by experience with plasticity characteristics as for material Type 1.

Table 11-2

Test Sieve Size(mm)	Percent(%) by mass of total aggregate passing test sieve					
	Type 1	Type 2	Type 3	Type 4	Type 5	Type 6
50	-	-	-	100	-	-
37.5	100	-	100	80-100	-	-
28	-	100	95 - 100	-	-	-
20	80 - 100	95 - 100	85-100	60-80	100	-
14	-	80-100	65 - 100	-	-	-
10	55 - 100	65 - 100	55 - 100	45-65	80 - 100	100
5	40 - 60	45 - 85	35-90	30-50	60 -85	80-100
2.36	30 - 50	-	-	20-40	45-70	50-80
2	-	30 - 65	22-75	-	-	-
1	-	25-55	18-60	-	-	-
0.425	15 - 30	18 - 45	15-50	10-25	25-45	25-45
0.075	5 - 15	12-32	10-40	5-15	10-25	10-25

11.7.3 Major Gravel Roads (AADTdesign = 20 to 200)

Major gravel roads are roads which have a design AADT greater than 20 and less than 200. These will generally fall within the design category of DS5 to DS8 (See ERA *Geometric Design Manual -2002*, Chapter 2. It is recommended to use a gravel wearing course material of grading Type 1 in the new construction of roads having an AADT greater than 50 and for all routine and periodic maintenance activities. Type 4 material may be used in the new construction of roads having an AADT less than 50.

11.7.4 Minor Gravel Roads (AADTdesign < 20)

Minor gravel roads are roads which have a design AADT ($AADT_{design}$) less than 20. They are normally community roads, which are constructed by labor-based methods. These roads generally fall within the design category of DS9 to DS10 (Refer to ERA *Geometric Design Manual-2002*). Usually these roads are unsurfaced (earth roads). However, for subgrade CBR values less than 5% and longitudinal gradients of greater than 6%, a gravel wearing course is recommended. Materials for gravel wearing course shall comply with the requirements for Type 4 material for new construction and Type 1 for maintenance activities.

The CBR requirements may be reduced to 20% if other suitable material is not locally available.

11.8 Determination of CBR_{design}

11.8.1 General

The CBR_{design} is the CBR value of a homogenous section, for which the subgrade strength is classified into S5, S4 or S2 for the purpose of pavement design. The procedure to determine CBR_{design} is shown in the flow chart in Figure 11-2.

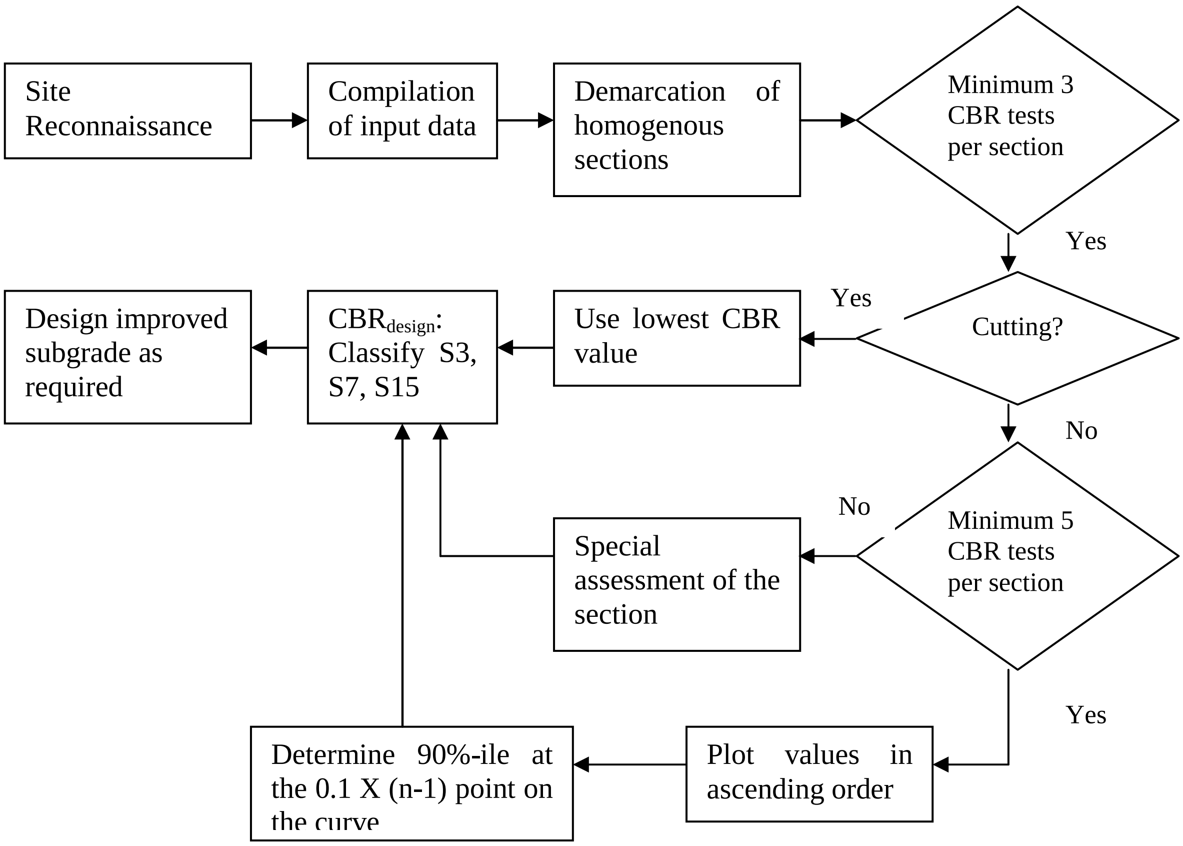


Figure 11-2: Flow Chart for Design

11.8.2 Homogenous Sections

Identification of sections deemed to have homogenous subgrade conditions is carried out by desk studies of appropriate documents such as geological maps, followed by site reconnaissance that includes excavation of inspection pits and initial indicator testing for confirmation of the site observations. Due regard for localized areas that require individual treatment is an essential part of the site reconnaissance. Demarcation of homogenous sections shall be reviewed and changed as required when the CBR test results of the centerline soil survey are available.

11.8.3 Statistical Analysis

The flow chart in Figure 2 shows the procedure to determine CBR_{design}.

The CBR_{design} for cuttings is the lowest CBR value encountered for the homogenous section.

The CBR_{design} for sections that do not require special assessment or are not within cuttings are determined by the 90%-ile value of the CBR test results. The 90%-ile value for a section of this type is the CBR value which 10% of the test results fall below. The following example shows how this is calculated.

1. CBR values are plotted in ascending order (number of tests on the "x axis" and the CBR test result values on the "y axis");

2. Calculate d = 0.1 x (n-1), where n = number of tests;

3. d is measured along the "x axis" and the CBR_{design} is determined from the "y axis".

11.8.4 Laboratory Testing

Each CBR value shall be determined by laboratory measurement carried out for a minimum of three density values to give a CBR - Density relationship for the material. The CBR value is determined at the normal field density specified for the respective operation (i.e. a minimum in-situ density of 95% of the maximum dry density determined in accordance with the requirements of AASHTO T 180).

11.9 Improved Subgrade and Pavement Design

11.9.1 Major Gravel Roads

Pavement and improved subgrade for major gravel roads shall be constructed in accordance with Figure 11-3. This includes all design categories DS5, DS6, DS7 and DS8 as defined in ERA *Geometric Design Manual -2002*.

11.9.2 Minor Gravel Roads

Pavement and improved subgrade for minor gravel roads shall also be constructed in accordance with Figure 11-3. This includes design categories DS9 and DS10 as defined in ERA *Geometric Design Manual -2002*. The desired properties of the gravel wearing course material, GW, are given in Section 11.7. However, the CBR may be reduced to 20%, and the LA abrasion value may be increased to 55% for minor roads, if better quality material is not locally available.

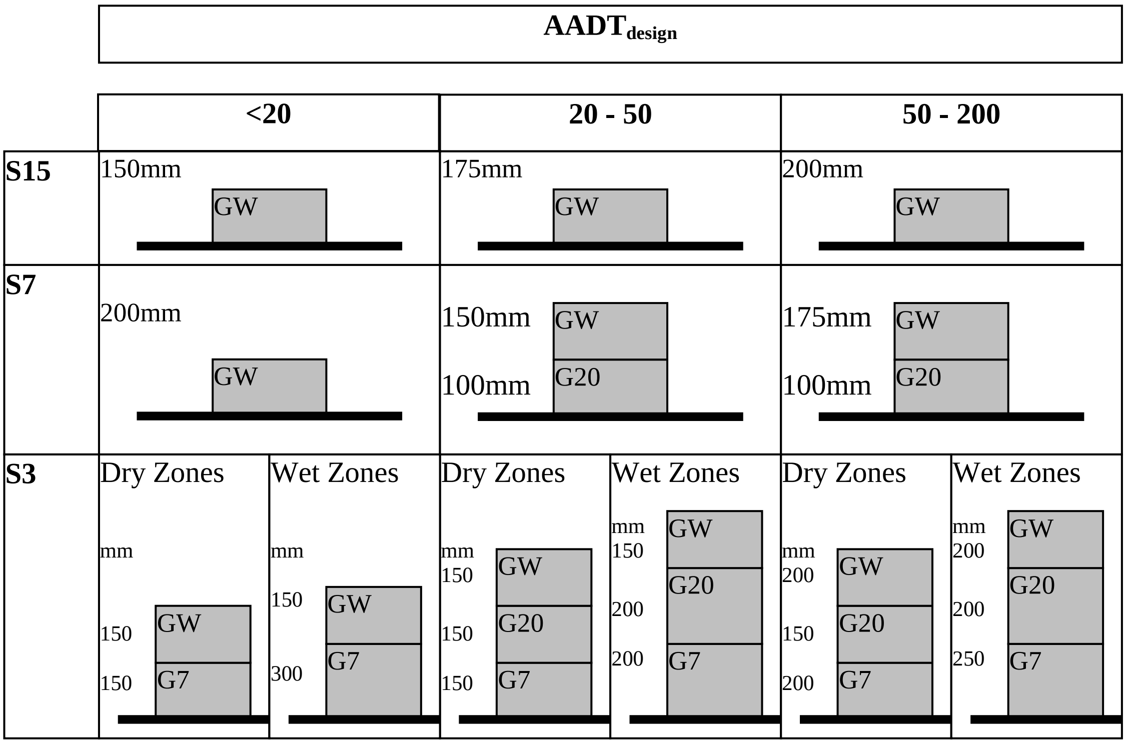


Figure 11-3: Pavement and Improved Subgrade for Gravel Roads for ADDTs < 200

11.10 Climatic Zones

11.10.1 Zones

For the purposes of gravel wearing course design, Ethiopia is divided into two climatic zones. All places with elevations over 2,000 meters (average rainfall 80mm/month) are considered to be wet zones and all places with elevations 2,000 meters or less (average rainfall 20mm/month) are considered to be dry zones. However, engineering judgement should be made for individual projects as to which category the design falls.

11.10.2 Arid Areas

It is acknowledged that, in many arid areas, rates of rainfall may be extremely high over short durations. Pavement design techniques, unlike drainage design techniques, do not take this into account as they are based on annual rates of rainfall.

APPENDIX A APPLICABLE STANDARDS

Designation	**Name of Test**
Tests on Soils	
ASTM D698	Moisture-Density Relations of Soils/Soil-Aggregate Mix (Light)
ASTM D1557	Moisture-Density Relations Using Modified Effort (Heavy)
AASHTO T180	Compaction Test
ASTM D1883 or AASHTO T193	CBR Test for Laboratory-Compacted Soils
ASTM C136 or AASHTO T27	Particle Size Distribution
ASTM D424 or AASHTO T90	Plastic Limit and Plasticity Index
ASTM D2487 or AASHTO M145	Classification of Soils for Engineering Purposes
Tests on Gravels	
ASTM D698	Moisture-Density Relations of Soils/Soil-Aggregate Mix (Light)
ASTM D1557	Moisture-Density Relations Using Modified Effort
AASHTO T180	Compaction Test
ASTM D1883 or AASHTO T193	CBR Test for Laboratory-Compacted Soils
ASTM C136 or AASHTO T27	Particle Size Distribution
ASTM D4318	Liquid Limit
ASTM D424 or AASHTO T90	Plastic Limit and Plasticity Index
BS 812, Part 105	Flakiness Index (see also ASTM Practice C 670)
BS 812, Part 111	Ten Per Cent Fines Value (TFV)
ASTM C535-89	Los Angeles Abrasion Test (LAA)
BS 812, Part 110	Aggregate Crushing Value (ACV)
BS 812, Part 112	Aggregate Impact Test
ASTM C88	Sodium Soundness Test (SST)
BS 1924, Part 2	Initial Consumption of Lime (ICL)
ASTM C977	Lime for Soil Stabilization
Tests on Asphalts and Bitumens	
BS 812, Part 103	Cleanliness (Sedimentation or Decantation)
BS 812, Part 105	Flakiness Index (see also ASTM Practice C 670)
BS 812, Part 110	Aggregate Crushing Value (ACV)
ASTM C131 and ASTM C535-89	Los Angeles Abrasion Test (LAA)
BS 812, Part 110	Polished Stone Value
ASTM C88	Sodium Soundness Test (SST)

ASTM C88	Magnesium Soundness Test (SST)
ASTM C127	Water Absorption for Fines
ASTM C128	Water Absorption for Coarse Materials
AASHTO T-182	Bitumen Affinity (Coating and Stripping)
AASHTO T-176	Sand Equivalent
ASTM D4318	Plastic Limit and Plasticity Index
ASTM D2872	Rolling Thin Film Oven Test (TFOT)
ASTM D1559 or AASHTO T245	Marshall Mix Design and Test
ASTM D4402	Viscosity Determination
ASTM D5	Penetration of Bituminous Materials
ASTM D3910	Slurry Seal Design and Testing
TRL ORN 31	Refusal Density Mix Design
TRL ORN 3	DCP Test
TRL ORN 3	Immersion Tray Test
TRL ORN 3	Probe Penetration Test
ASTM D3497	Dynamic Modulus of Bituminous Material

APPENDIX B ESTIMATING SUBGRADE MOISTURE CONTENT FOR CATEGORY 1 CONDITIONS

The subgrade moisture content under an impermeable road pavement can increase after construction where a water table exists close to the ground surface. This ultimate moisture content can be predicted from the measured relationship between soil suction and moisture content for the particular soil and knowledge of the depth of water table.

Measuring the complete relationship between suction and moisture content is time consuming and a simpler, single measurement procedure can be used. A small sample of soil, compacted to field density and moisture content, is placed within suitable laboratory equipment that can apply a pressure equivalent to the effective depth of the water table (e.g. a pressure plate extractor). The effective depth of the water table for design purposes comprises the actual depth from the subgrade to the water table plus an apparent depression of the water table due to the pressure of the overlying pavement. This apparent depression varies with soil type and an approximate correction factor is given in Table B-1.

Table B-1: Correction Factors for Soil Type P1 Used in Calculating the Effective Depth of the Water Table

P1	Correction factor SF
0	0
10	0.3
15	0.55
20	0.80
25	1.1
30	1.4
35	1.6
>35	2.0

To calculate the effective depth D which is used to determine the applied suction in the pressure plate extractor, the following equation is used:

$$D = WT + (SF \times t)$$

Where WT = depth of water table below subgrade (at its highest expected seasonal level).
SF = correction factor from Table B-1.
t = pavement thickness, with consistent units for WT, t, and D.

When equilibrium is attained in the pressure plate extractor, the sample is removed and its moisture content measured. This moisture content is the value at which the CBR for design should be estimated following standard soil tests as outlined in Section 3.2.

APPENDIX C TRL DYNAMIC CONE PENETROMETER

The TRL Dynamic Cone Penetrometer (DCP), shown in Figure C-l, is an instrument designed for the rapid in situ measurement of the structural properties of existing road pavements with unbound granular materials. Continuous measurements can be made to a depth of 800 mm or to 1200 mm when an extension rod is fitted.

The underlying principle of the DCP is that the rate of penetration of the cone, when driven by a standard force, is inversely related to the strength of the material as measured by, for example, the California Bearing Ratio (CBR) test (see Figure C-2). Where the pavement layers have different strengths, the boundaries between the layers can be identified and the thickness of the layers determined. A typical result is shown in Figure C-3.

The DCP needs three operators; one to hold the instrument, one to raise and drop the weight and a technician to record the results. The instrument is held vertical and the weight carefully raised to the handle. Care should be taken to ensure that the weight is touching the handle, but not lifting the instrument, before it is allowed to drop and that the operator lets it fall freely and does not lower it with his hands. If during the test the DCP tilts from the vertical, no attempt should be made to correct this as contact between the shaft and the sides of the hole will give rise to erroneous results. If the angle of the instrument becomes worse, causing the weight to slide on the hammer shaft and not fall freely, the test should be abandoned.

It is recommended that a reading should be taken at increments of penetration of about 10 mm. However it is usually easier to take readings after a set number of blows. It is therefore necessary to change the number of blows between readings according to the strength of the layer being penetrated. For good quality granular base courses readings every 5 or 10 blows are normally satisfactory but for weaker sub-base layers and subgrade readings every 1 or 2 blows may be appropriate.

Little difficulty is normally experienced with the penetration of most types of granular or weakly stabilized materials. It is more difficult to penetrate strongly stabilized layers, granular materials with large particles and very dense, high quality crushed stone. The TRL instrument has been designed for strong materials and therefore the operator should persevere with the test. Penetration rates as low as 0.5 mm/blow are acceptable but if there is no measurable penetration after 20 consecutive blows it can be assumed that the DCP will not penetrate the material. Under these circumstances a hole can be drilled through the layer using either an electric or pneumatic drill or by coring. The lower layers of the pavement can then be tested in the normal way.

DCP results are conveniently processed by computer and a program has been developed that is designed to assist with the interpretation and presentation of DCP data.

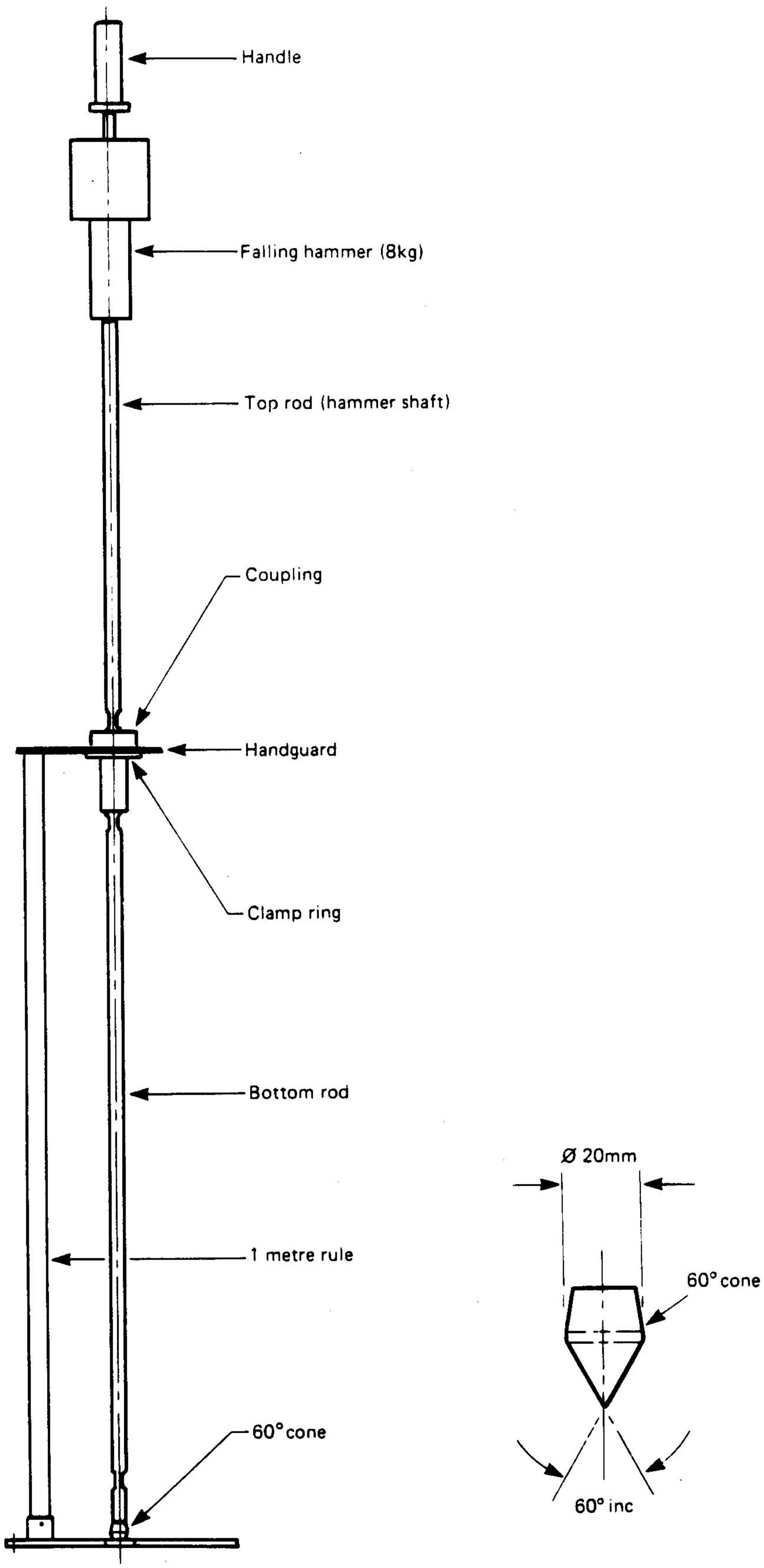


Figure C-1: TRL Dynamic Cone Penetrometer

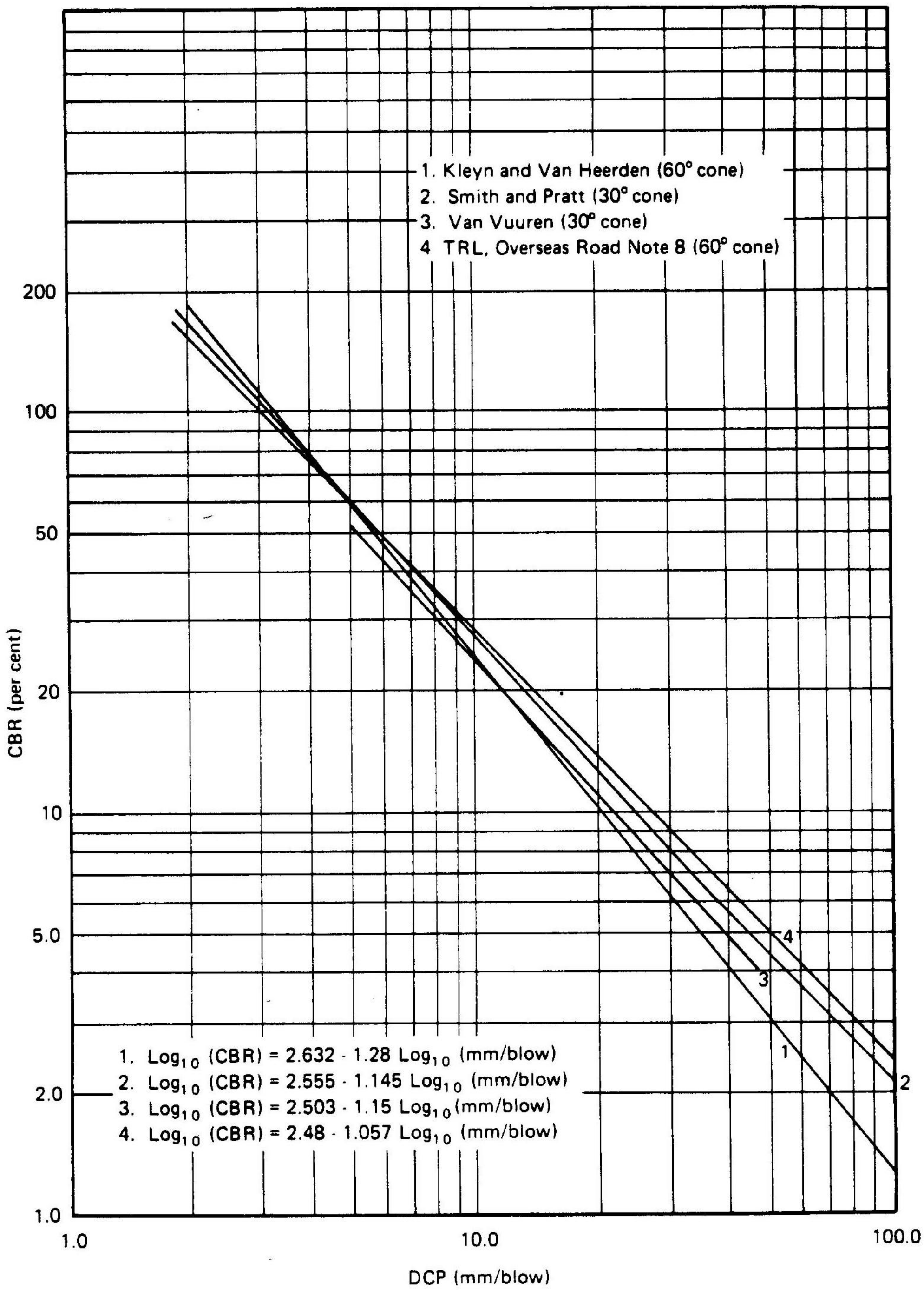


Figure C-2: DCP-CBR Relationships

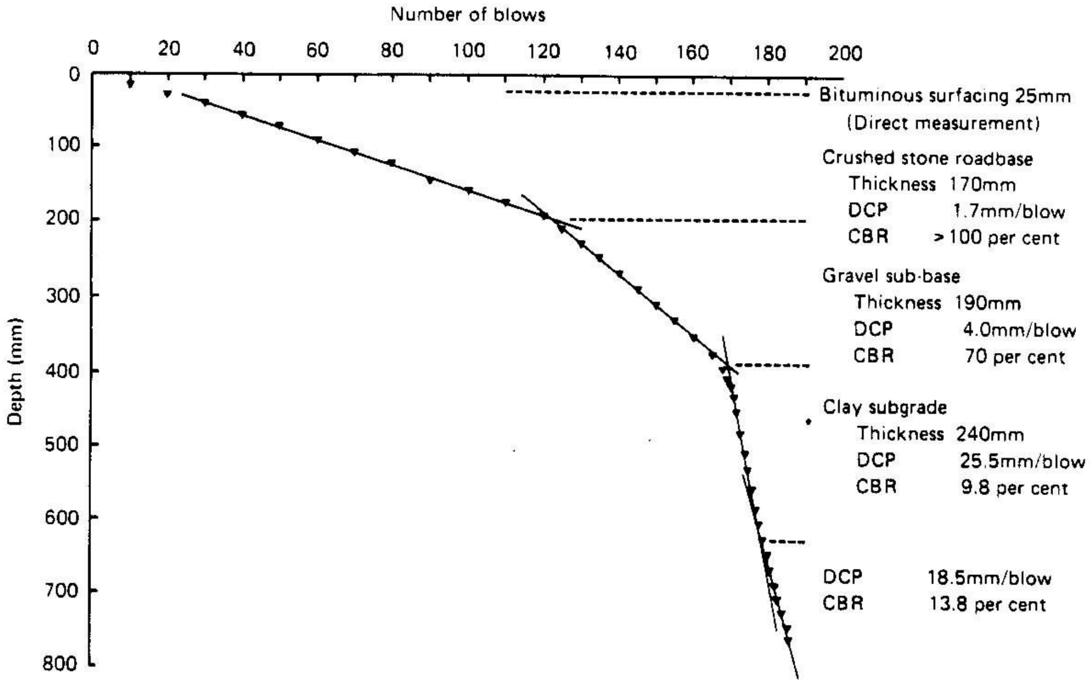


Figure C-3: DCP Test Result

APPENDIX D REFUSAL DENSITY DESIGN

D1. Introduction

The extended Marshall compaction procedure can be used to design asphalts which will retain a required minimum voids in the mix (VIM) after secondary compaction by traffic. An alternative method based on an extended form of the compaction procedure used in the Percentage Refusal Density (PRD) Test (BS 598 Part 104 (1989)) uses a vibrating hammer for compaction. These methods are appropriate for sites which are subject to severe loading where research shows that it is desirable to retain a minimum VIM of three per cent to minimise the risk of failure by plastic deformation. Neither method exactly reproduces the mode of compaction which occurs under heavy traffic but the latter procedure is both quicker and more representative. There are no national or international standards for these procedures and therefore they are both likely to be subject to further development.

D2. Extended Marshall Compaction

For severe sites, the basecourse specifications, BC1 and BC2, given in Table 8-4 and Table 8-7, are likely to be the most appropriate. The normal Marshall design procedure using 75 blows on each face should be completed first to provide an indication that the Marshall design parameters will be met.

The binder content corresponding to 6 per cent VIM obtained in the Marshall test should be noted and additional test samples prepared at each of three binder contents, namely the binder content corresponding to 6 per cent VIM and also binder content which are 0.5 per cent above and 0.5 per cent below this value. These samples must be compacted to refusal.

The number of blows required to produce a refusal condition will vary from one mix to another. It is preferable to conduct a trial using the lowest binder content and to compact using an increasing number of blows, say 200, 300, 400, etc. until no further increase in density occurs. Usually 500 blows on each face is found to be sufficient.

By plotting a graph of VIM at the refusal density against binder content the design binder content which corresponds to a VIM of 3 per cent can be determined. This value should be obtained by interpolation, not by extrapolation. If necessary, the binder content range should be extended upwards or downwards, as appropriate, to permit this.

D3. Extended Vibrating Hammer Compaction

LABORATORY DESIGN PROCEDURE

In the vibrating hammer method, the samples are compacted in 152-153 mm diameter moulds to a thickness approximately the same as will be laid on the road. The BS 598 compaction procedure for the PRD test is repeated if necessary to achieve an absolute refusal density. The electric vibrating hammer should have a power consumption of 750 watts or more and operate at a frequency of 20 to 50 Hz. Two tamping feet are used, one with a diameter of 102 mm and the other of 146 mm. Samples should be mixed so that they can be compacted immediately afterwards at an initial temperature of 140 ± 5°C for 80/100 penetration grade bitumen or 145 ± 5°C for 60/70 penetration grade bitumen.

The moulds and tamping feet must be pre-heated in an oven before starting the test. Cooling of the sample by as much as 15 to 20°C during compaction should not prevent achievement of the correct refusal density. The small tamping foot is used for most of the compaction sequence. The hammer must be held firmly in a vertical position and moved from position to position in the prescribed order, i.e. using the points of a compass. To identify the position, the order should be N, S, W, NW, SE, SW, NE or equivalent. At each point, compaction should continue for between 2 and 10 seconds, the limiting factor being that material should not be allowed to push up around the compaction foot. The compaction sequence is continued until a total of 2 minutes ± 5 seconds of compaction time has been reached. The large tamping foot is then used to smooth the surface of the sample.

A spare base-plate, previously heated in the oven, is placed on top of the mould which is then turned over. The sample is driven to the new base plate with the hammer and large tamping foot. The compaction sequence is then repeated. The free base plate should be returned to the oven between compaction cycles.

This is the standard PRD compaction procedure but to ensure that the refusal density is reached, it may be necessary to repeat this procedure a second time. It is suggested that trial mixes with a bitumen content which corresponds to approximately 6 per cent VIM in the Marshall test, are used to

(i) Determine the mass of material required to give a compacted thickness of approximately the same thickness as for the layer on the road.

(ii) Determine the number of compaction cycles which will ensure that absolute refusal density is achieved.

After these tests have been completed, samples are made with bitumen contents starting at the Marshall optimum and decreasing in 0.5 per cent steps until the bitumen content at which 3 per cent voids is retained at absolute refusal density can be determined.

TRANSFER OF LABORATORY DESIGN TO COMPACTION TRIALS

After the standard PRD compaction cycle, test samples of basecourse or roadbase which have been compacted from the loose state can be expected to have densities between 1.5 and 3 per cent lower than for the same material compacted in the road but cored out and subjected to the PRD test. This is an indication of the effect of the different compaction regimes and is caused by a different resultant orientation of particles. The differences between the densities for laboratory and field samples after refusal compaction should be measured to confirm whether this difference occurs.

A minimum of three trial lengths should be constructed with bitumen contents at the laboratory optimum for refusal density (93 per cent VIM) and at 0.5 per cent above and 0.5 per cent below the optimum. These trials should be used to:

(i) Determine the rolling pattern required to obtain a satisfactory density

(ii) Establish that the mix has satisfactory workability to allow a minimum of 93 per cent of PRD (standard compaction (BS598: 1989)) to be achieved after rolling

(iii) Obtain cores so that the maximum binder content which allows at least 3 per cent VIM to be retained at refusal density can be confirmed.

For a given aggregate and grading, cores cut from the compacted layer can be expected to give a constant value of voids in the mineral aggregate (VMA) at the refusal density, irrespective of bitumen content. This will allow a suitable binder content to be chosen to give a minimum of 3 per cent VIM at refusal density.

A minimum of 93 per cent and a mean value of 95 per cent of the standard PRD density is recommended as the specification for density on completion of compaction of the layer. From these trials and the results of laboratory tests, it is then possible to establish a job mix formula. This initial procedure is time consuming, but is justified by the long term savings in extended pavement service life that can be obtained. After this initial work, subsequent compliance testing based on analysis of mix composition and refusal density should be quick, especially if field compaction can be monitored with a nuclear density gauge.

It is essential to provide a surface dressing for the type of basecourse mixes which are best suited to these severe conditions. This protects the mix from severe age hardening during the period when secondary compaction occurs in the wheelpaths, and also protects those areas which will not be trafficked and are likely to retain air voids above 5 per cent.

D4. Possible Problems with the Test Procedures

Multi-blow Marshall compaction and vibratory compaction may cause breakdown of aggregate particles. If this occurs to a significant extent then the test is unlikely to be valid.

Because of the time taken to complete the Marshall procedure, considerable care must be taken to prevent excessive cooling of the sample during compaction.

It is important to note that the different particle orientation produced by these compaction methods, in comparison with that produced by roller compaction, limits the use of samples prepared in these tests to that of determining VIM at refusal. It would be unwise to use samples prepared in this way for fatigue or creep tests.

APPENDIX E THE IMMERSION TRAY TEST FOR DETERMINING THE CONCENTRATION OF ADHESION AGENT REQUIRED

The following test procedure has been included in editions of Road Note 39, Design Guide for Road Surface Dressing (Ref. 19) since at least 1964. The method is reproduced below and then suggestions are made which may help to make it more appropriate for Ethiopia.

In this test a tin lid approximately 135mm diameter is covered with 15 to 20g of binder giving a film some 1.5mm thick. When this has cooled to the test temperature* it is immersed in water also at test temperature to a depth of about 25mm. Nominal 14mm chippings are then applied by hand and lightly pressed in. At least six pieces of the aggregate are used. The chippings are left for 10 minutes and are then carefully removed from the binder film: the percentage of binder retained on the chippings is assessed visually.

When testing an adhesion agent, a known quantity of agent is added to the binder and thoroughly stirred to ensure good dispersion. The procedure is then as outlined above. The test is repeated with varying concentrations of agent in the binder until the minimum concentration required to give satisfactory results has been found. The concentration normally falls in the range 0.5 to 2.5 per cent by mass of agent.

The agent may be considered satisfactory for use on the road if, in the test, when the chippings are lifted from the binder film the faces which have been in contact with the film are all 90-100 per cent coated with binder.

***The temperature of water and tray of binder in the above test should be the expected temperature of the road surface during the treatment. Where it is desired to compare the behavior of different agents with a given stone and binder it is suggested that 20^{o}C should be used as the test temperature.**

SUGGESTED NEW PROCEDURE

For Ethiopian conditions the test bitumen should be of the grade to be used on site and it should be tested at appropriate site temperatures. Testing different adhesion agents at 20°C is not practical if, for instance, hot conditions warrant the use of an 80/100 penetration grade bitumen. It is considered appropriate to test the adhesion agents at a temperature which relates to the design road temperature on which binder selection was based.

A tin lid approximately 135mm diameter, or other suitable tray, is covered with binder to give a film some 1.5mm thick. Place at least 10 chippings which are damp, but not with shiny wet surfaces, in the film of binder at the “design road temperature” and leave for 10 minutes. Then withdraw some of the chips to confirm coating. Add water to about half the depth of the remaining chippings at the chosen test temperature and leave for 10 minutes before withdrawing them and noting the degree of coating. If the coating is less than 90 per cent on any chipping then an adhesion agent should be tried. In this case different percentages of the adhesion agent are added to samples of the binder until 90-100 per cent coverage is obtained, after soaking, on all chippings.

If limestone chippings are available they will provide a good comparison of adhesion properties with the chippings to be used on site because limestone has good affinity with bitumen.

APPENDIX F THE PROBE PENETRATION TEST FOR MEASURING ROAD SURFACE HARDNESS

F1. General Description

This test utilizes a modified soil assessment cone penetrometer, originally designed by the UK Military Engineering Experimental Establishment for the assessment of in-situ soil strength. The standard cone normally used with this penetrometer is replaced by a 4mm diameter probe rod with a hemispherical tip made of hardened steel. The probe is forced into the road surface under a load of 35kgf (343N) applied for 10 seconds and the depth of penetration is measured by a spring loaded collar that slides up the probe rod. The distance the collar has moved is measured with a modified dial gauge. The temperature of the road surface is recorded and a graphical method is used to correct the probe measurements to an equivalent value at a standard temperature of 30°C.

F2. Method of Operation

All measurements are made in the nearside wheel track of each traffic lane where maximum embedment of chippings can be expected. A minimum of ten measurements is required at each location. These should be evenly spaced along the road at intervals of 0.5m, any recently repaired or patched areas being ignored. For convenience the measurement points can be marked with a chalk cross. The probe tip should not be centered on any large stones present in the road surface.

Before each measurement the collar is slid down the probe rod until it is flush with the end of the probe. The probe is then centered on the measurement mark and a pressure of 35kgf is applied for 10 seconds, care being taken to keep the probe vertical. The probe is then lifted clear and the distance the collar has slid up the probe is recorded in millimeters.

It sometimes occurs that the point selected for test is below the general level of the surrounding road surface. It is then necessary to deduct the measurement of the initial projection of the probe tip from the final figure.

The road surface temperature should be measured at the same time that the probe is used and the tests should not be made when the surface temperature exceeds 35°C. This will limit probe testing to the early morning in many locations. The probe readings are corrected to a standard temperature of 30°C using Figure F-1, and the mean of ten probe measurements is calculated and reported as the mean penetration at 30°C. Categories of road surface hardness and the corresponding ranges of surface penetration values are shown in Table 9-7.

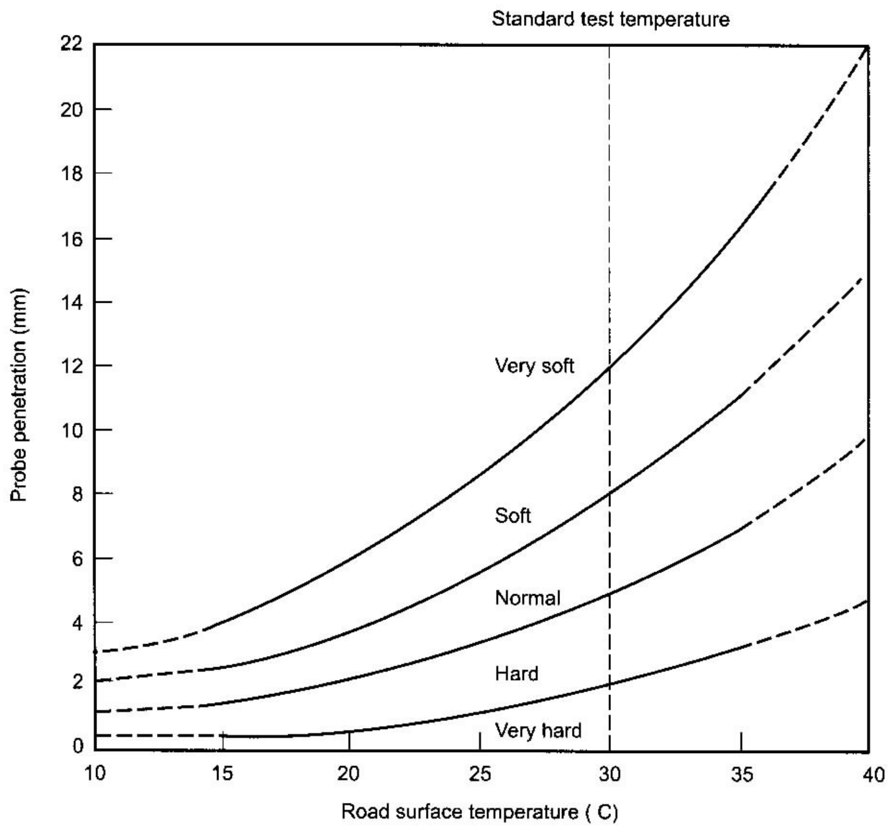


Figure F-1: Graphical Method for Correcting Measurements of Road Surface Hardness to the Standard Test Temperature of 30^{o}C

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