PREFACE

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

The Roads Development Agency has prepared this Manual under a credit from the International Labour Organization (ILO) for design of roads in order to standardize design practices in all RDA 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.

The design standards set out in this Manual shall be adhered to unless otherwise directed by the concerned bodies with in RDA. 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 “Pavement Design Manual –2014.”

It is my sincere hope that this Manual will provide all users with both a standard reference and a ready source of good practice for the Pavement design, 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.

ROADS DEVELOPMENT AGENCY
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1 INTRODUCTION

1.1 GENERAL

This manual gives recommendations for the structural design of flexible pavement and gravel roads in Somaliland. 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 Somaliland.

1.2 UNDERLYING PRINCIPLES

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.

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

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, \( P_0 \). 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.

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

Figure 1-2: Elements of a Flexible Pavement
**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.

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**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).

**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](image)

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](image)

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-2014);
- 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

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 certain conservatism is justified. Moreover, sensitivity analyses of the resulting pavement structures to these parameters are recommended.

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 Somaliland where a wide range of climatic zones are encountered, from maritime semidesert plain parallels the
Gulf of Aden coast, to temperate and mountainous (subalpine) over a significant part of the country, with annual rainfalls up to 600 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).

Figure 1-6: Pavement Design Process
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.

**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.

**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).

**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.

**Main 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.

**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.

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 RDA’s Pavement Management System (PMS) and is expected to evolve and be refined as the PMS procedures are implemented in Somaliland. 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.
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.
Select Design Period

Estimate Initial Traffic Volume (Initial AADT) per Class of Vehicle

Estimate Traffic Growth

Determine Cumulative Traffic Volumes over the Design Period

For Flexible Pavements

Estimate Mean Equivalent Axle Load (ESA) per Class of Vehicle

Estimate Cumulative ESAs Over the Design Period (in one direction)

Select Appropriate Traffic Class (based on ESAs) for Flexible Pavement Design

For Gravel Roads

Select Appropriate AADT for Design of Gravel Wearing Course

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

**Table 2-1: Design Period**

<table>
<thead>
<tr>
<th>Road Classification</th>
<th>Design Period (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trunk Road</td>
<td>20</td>
</tr>
<tr>
<td>Link Road</td>
<td>20</td>
</tr>
<tr>
<td>Main Access Road</td>
<td>15</td>
</tr>
<tr>
<td>Other Roads</td>
<td>10</td>
</tr>
</tbody>
</table>

2.3 **Traffic Volumes**

**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 RDA 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**

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

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.
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 Somaliland, 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.

**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.

**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 \( (\text{AADT}_0) \) 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 \( \text{"i"} \) expressed as a decimal fraction, and the anticipated number of years \( \text{"x"} \) between the traffic survey and the opening of the road.

3. Determine \( \text{AADT}_1 \) the traffic volume in both directions on the year of the road opening by:

\[
\text{AADT}_1 = \text{AADT}_0 \ (1+i)^x
\]
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:

$$T = 365 \text{ AADT}_1 \left[ \frac{(1+i)^N - 1}{i} \right] / (i)$$

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

2.4 AXLE LOADS

**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.

**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 Somaliland, 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 Somaliland 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.

**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:

\[
\text{Equivalency factor} = \left( \frac{\text{Axle mass}}{8160} \right)^n
\]

where

- \( \text{axle}_i \) = 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 is given in Table 2-3:

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

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

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 Chapter 13 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 Somaliland. 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>
<thead>
<tr>
<th>Number of lanes in each direction</th>
<th>Percent of ESAs in design lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100</td>
</tr>
<tr>
<td>2</td>
<td>80 – 100</td>
</tr>
<tr>
<td>3</td>
<td>60 – 80</td>
</tr>
</tbody>
</table>

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>
<thead>
<tr>
<th>Traffic classes</th>
<th>Range (10^6) ESAs</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>(&lt; 0.3)</td>
</tr>
<tr>
<td>T2</td>
<td>0.3 - 0.7</td>
</tr>
<tr>
<td>T3</td>
<td>0.7 - 1.5</td>
</tr>
<tr>
<td>T4</td>
<td>1.5 - 3.0</td>
</tr>
<tr>
<td>T5</td>
<td>3.0 - 6.0</td>
</tr>
<tr>
<td>T6</td>
<td>6.0 - 10</td>
</tr>
<tr>
<td>T7</td>
<td>10 - 17</td>
</tr>
<tr>
<td>T8</td>
<td>17 – 30</td>
</tr>
</tbody>
</table>
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 2014 for a section of a trunk road under study, as follows:

<table>
<thead>
<tr>
<th>Vehicle classification</th>
<th>2014 AADT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Car</td>
<td>250</td>
</tr>
<tr>
<td>Bus</td>
<td>40</td>
</tr>
<tr>
<td>Truck</td>
<td>130</td>
</tr>
<tr>
<td>Truck-trailer</td>
<td>180</td>
</tr>
</tbody>
</table>

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):
The projected AADTs in 2005 can be calculated as \((\text{AADTs in 2001}) \times (1.05)^3\), and the corresponding one-directional volumes for each class of vehicle in 2005 are:

<table>
<thead>
<tr>
<th>Vehicle classification</th>
<th>One-directional traffic volume in 2014</th>
</tr>
</thead>
<tbody>
<tr>
<td>Car</td>
<td>145</td>
</tr>
<tr>
<td>Bus</td>
<td>23</td>
</tr>
<tr>
<td>Truck</td>
<td>75</td>
</tr>
<tr>
<td>Truck-trailer</td>
<td>104</td>
</tr>
</tbody>
</table>

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:

<table>
<thead>
<tr>
<th>Vehicle classification</th>
<th>Cumulative no. of vehicles in one direction over 20 years</th>
</tr>
</thead>
<tbody>
<tr>
<td>Car</td>
<td>365\times145[(1.05)^{20}-1]/0.05=1750016</td>
</tr>
<tr>
<td>Bus</td>
<td>365\times23[(1.05)^{20}-1]/0.05=277589</td>
</tr>
<tr>
<td>Truck</td>
<td>365\times75[(1.05)^{20}-1]/0.05=905180</td>
</tr>
<tr>
<td>Truck-trailer</td>
<td>365\times104[(1.05)^{20}-1]/0.05=1255184</td>
</tr>
</tbody>
</table>

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:

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

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:

<table>
<thead>
<tr>
<th>Vehicle classification</th>
<th>Cum. no. of vehicles</th>
<th>Equivalency factor</th>
<th>$10^6$ ESAs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Car</td>
<td>1750016</td>
<td>0.00</td>
<td>0.0</td>
</tr>
<tr>
<td>Bus</td>
<td>277589</td>
<td>0.14</td>
<td>0.0</td>
</tr>
<tr>
<td>Truck</td>
<td>905180</td>
<td>6.67</td>
<td>6.0</td>
</tr>
<tr>
<td>Truck-trailer</td>
<td>1255184</td>
<td>11.47</td>
<td>14.4</td>
</tr>
</tbody>
</table>

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 Somaliland. In such circumstances, the default values given in Table 2.6 should be used:

<table>
<thead>
<tr>
<th>Axles per Heavy Vehicle</th>
<th>2.3</th>
</tr>
</thead>
<tbody>
<tr>
<td>ESAs per Heavy Axle</td>
<td>0.20</td>
</tr>
<tr>
<td>ESAs per Heavy Vehicle</td>
<td>0.46</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>First Year Traffic</th>
<th>Cumulative Values after 10 Years</th>
<th>Cumulative Values after 15 Years</th>
<th>Cumulative Values after 20 Years</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Heavy vehicles per lane</td>
<td>Heavy vehicles per lane</td>
<td>ESAs</td>
</tr>
<tr>
<td>AADT</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>3</td>
<td>13 803</td>
<td>6 350</td>
</tr>
<tr>
<td>20</td>
<td>4</td>
<td>15 337</td>
<td>7 055</td>
</tr>
<tr>
<td>30</td>
<td>5</td>
<td>22 786</td>
<td>10 482</td>
</tr>
<tr>
<td>40</td>
<td>7</td>
<td>30 674</td>
<td>14 110</td>
</tr>
<tr>
<td>50</td>
<td>9</td>
<td>39 438</td>
<td>18 141</td>
</tr>
<tr>
<td>60</td>
<td>11</td>
<td>46 011</td>
<td>21 165</td>
</tr>
<tr>
<td>80</td>
<td>14</td>
<td>61 348</td>
<td>28 220</td>
</tr>
<tr>
<td>100</td>
<td>18</td>
<td>76 685</td>
<td>35 275</td>
</tr>
<tr>
<td>120</td>
<td>21</td>
<td>92 022</td>
<td>42 330</td>
</tr>
<tr>
<td>150</td>
<td>26</td>
<td>113 932</td>
<td>52 409</td>
</tr>
<tr>
<td>180</td>
<td>32</td>
<td>138 033</td>
<td>63 495</td>
</tr>
<tr>
<td>200</td>
<td>35</td>
<td>155 379</td>
<td>70 050</td>
</tr>
<tr>
<td>300</td>
<td>55</td>
<td>230 055</td>
<td>105 825</td>
</tr>
<tr>
<td>350</td>
<td>61</td>
<td>268 398</td>
<td>123 463</td>
</tr>
<tr>
<td>400</td>
<td>70</td>
<td>306 740</td>
<td>141 100</td>
</tr>
<tr>
<td>500</td>
<td>88</td>
<td>383 425</td>
<td>176 376</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Vehicle Type</th>
<th>Average ESAs per Vehicle</th>
<th>Typical Range of Average ESAs per Vehicle</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-axle truck</td>
<td>0.70</td>
<td>0.30 – 1.10</td>
</tr>
<tr>
<td>2-axle bus</td>
<td>0.73</td>
<td>0.41 – 1.52</td>
</tr>
<tr>
<td>3-axle truck</td>
<td>1.70</td>
<td>0.80 – 2.60</td>
</tr>
<tr>
<td>4-axle truck</td>
<td>1.80</td>
<td>0.80 – 3.00</td>
</tr>
<tr>
<td>5-axle truck</td>
<td>2.20</td>
<td>1.00 – 3.00</td>
</tr>
</tbody>
</table>
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 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](image)
After the pavement is constructed, the moisture content of the subgrade will generally change.

In the dry southeast and northeast parts of Somaliland, 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.

Figure 3-2: Dry Density, Moisture Content, Soil Strength Relationship for a Well-Graded Sand
Chapter 3  Flexible Pavements and Gravel Roads

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 Somaliland, 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.
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.

**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. It remains nevertheless important to verify that the density assumed in design is consistent with the minimum density specified for a particular road project.

**SPECIFIC DENSITY-MOISTURE CONTENT-STRENGTH RELATIONSHIPS**

It is generally 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.

**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.

![Dry Density, Moisture Content-CBR Relationships for Sandy-Clay Soil](image)

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

The procedure outlined above 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, will require placement of an improved subgrade i.e. stabilization or removal and replacement with better quality material. Class S5 (CBR 15 to 29) and S6 (CBR 30 or more) soils will not require an improved subgrade.

### Table 3-1: Subgrade Strength Classes

<table>
<thead>
<tr>
<th>Class</th>
<th>Range (CBR %)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>2</td>
</tr>
<tr>
<td>S2</td>
<td>3 – 4</td>
</tr>
<tr>
<td>S3</td>
<td>5 – 7</td>
</tr>
<tr>
<td>S4</td>
<td>8 – 14</td>
</tr>
<tr>
<td>S5</td>
<td>15 – 29</td>
</tr>
<tr>
<td>S6</td>
<td>30+</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Depth of water table* from formation level (meters)</th>
<th>Subgrade strength class</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-plastic sand</td>
<td>Sandy clay PI=10</td>
</tr>
<tr>
<td>0.5</td>
<td>S4</td>
</tr>
<tr>
<td>1</td>
<td>S5</td>
</tr>
<tr>
<td>2</td>
<td>S5</td>
</tr>
<tr>
<td>3</td>
<td>S6</td>
</tr>
</tbody>
</table>

* 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 is 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.

**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 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 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 Somaliland. 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)

<table>
<thead>
<tr>
<th>Major Divisions</th>
<th>Symbol</th>
<th>Name</th>
<th>Value as Subgrade</th>
<th>Typical Design CBR Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>COARSE GRAINED SOIL</td>
<td>GW</td>
<td>Well-graded gravels or gravel-sand mixtures, little or no fines</td>
<td>Excellent</td>
<td>40-80</td>
</tr>
<tr>
<td></td>
<td>GP</td>
<td>Poorly graded gravels or gravel-sand mixtures, little or no fines</td>
<td>Good to excellent</td>
<td>30-60</td>
</tr>
<tr>
<td></td>
<td>GM</td>
<td>Silty gravels, gravel-sand-silt mixtures</td>
<td>Good to excellent</td>
<td>40-60</td>
</tr>
<tr>
<td></td>
<td>GC</td>
<td>Clayey gravels, gravel-sand-clay mixtures</td>
<td>Good</td>
<td>20-40</td>
</tr>
<tr>
<td>SAND AND SANDY SOILS</td>
<td>SW</td>
<td>Well-graded sands or gravelly sands, little or no fines</td>
<td>Good</td>
<td>20-40</td>
</tr>
<tr>
<td></td>
<td>SP</td>
<td>Poorly graded sands or gravelly sands, little or no fines</td>
<td>Fair to good</td>
<td>10-40</td>
</tr>
<tr>
<td></td>
<td>SM</td>
<td>Silty sands, sand-silt mixtures</td>
<td>Fair to good</td>
<td>15-40</td>
</tr>
<tr>
<td></td>
<td>SC</td>
<td>Clayey sands, sand-clay mixtures</td>
<td>Poor to fair</td>
<td>5-20</td>
</tr>
<tr>
<td>FINE GRAINED SOILS</td>
<td>ML</td>
<td>Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity</td>
<td>Poor to fair</td>
<td>15 or less</td>
</tr>
<tr>
<td></td>
<td>CL</td>
<td>Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays</td>
<td>Poor to fair</td>
<td>15 or less</td>
</tr>
<tr>
<td></td>
<td>OL</td>
<td>Organic silts and organic silt-clays of low Plasticity</td>
<td>Poor</td>
<td>5 or less</td>
</tr>
<tr>
<td>SILTS AND CLAYS LL IS GREATER THAN 50</td>
<td>MH</td>
<td>Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts</td>
<td>Poor</td>
<td>10 or less</td>
</tr>
<tr>
<td></td>
<td>CH</td>
<td>Inorganic clays of high plasticity, fat clays</td>
<td>Poor to fair</td>
<td>15 or less</td>
</tr>
<tr>
<td></td>
<td>OH</td>
<td>Organic clays of medium to high plasticity, Organic silts</td>
<td>Poor to very poor</td>
<td>5 or less</td>
</tr>
<tr>
<td>HIGHLY ORGANIC SOILS</td>
<td>Pt</td>
<td>Peat and other highly organic soils</td>
<td>Not suitable</td>
<td></td>
</tr>
</tbody>
</table>
NOTE: The division of GM and SM groups into subdivisions of d and u is on basis of Atterberg limits; suffix d (e.g., GMd) will be used when the liquid limit is 25 or less and the plasticity index is 5 or less; the suffix u will be used otherwise.
4 EARTHWORKS

4.1 INTRODUCTION

In this chapter, guidelines are given which pertain 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.

4.2 EMBANKMENTS

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.

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.
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.

**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 \(h \leq 1\ m\)
  - \(1: 2\) if \(h > 1\ m\)
- **Other materials:**
  - \(1: 3\) if \(h \leq 1\ m\)
  - \(1: 2\) 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 Somaliland, 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.
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.

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.

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 Somaliland, 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.

**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.

**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.

**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.

**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 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 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 RDA’s Geometric Design Manual-2014. 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. A more thorough discussion of expansive soils is given in Chapter 4.

Figure 5-1: Typical Pavement Cross Sections
5.6 EROSION CONTROL

Erosion problems may occur on the side slopes of embankments or cuttings, gravel shoulders or at any other point where surface run-off concentrated or a spring occurs. The obvious remedies are therefore well-designed surface and sub-surface drainage features and appropriate slope angles for the soils and rocks present. This last measure is problematic as there is no standard test to assess “erodibility”. The best guidance would be obtained from observations of actual road sections, assuming these exist.

Various surface protection systems can be used in conjunction with the above.

5.6.1 PROTECTION OF SLOPES

5.6.1.1 TOP SOILING AND GRASSING

Sprigs of indigenous “runner” type, grass may be planted on slopes by one or two methods:

The slope shall be covered with a layer of fine topsoil free of stones greater than 50mm. The minimum thickness should be 75mm. The layer shall then be planted with grass. Sprigs of grass shall be planted at approximately 200 mm centres in pockets of topsoil, 75 mm deep.

Planting should be carried out at the beginning of a rainy season.

5.6.1.2 SURFACE TREATMENTS WITH SEEDS AND FERTILIZERS

When difficulties are anticipated in establishing a healthy growth of grass on a sterile soil, a mixture of grass seeds and fertilizer may be applied. This can be done either as a wet or dry process. In the former process grass seed, fertilizer, mulch material and water are mixed to form slurry which is then sprayed onto the ground. In the dry process grass seed and fertilizer are mixed and applied to the ground, followed by watering and possible application of mulching material.

5.6.1.3 GRAVEL OR STONE BLANKETING

Erodible materials may be protected by placing coverings of gravel or stone blankets. The blanketing material should have a maximum size of 40 mm and he placed in an even layer of at least 75 mm.

5.6.1.4 FASCINES

Placing fascines or branches over the most vulnerable areas, generally combined with some form of grass planting, will help stabilize the slope until it is covered by grass or other vegetation.

5.6.1.4.1 SERRATED SLOPES

Serrated slopes aid in the establishment of vegetation. Serrations may be constructed in any material that is rippable or that will hold a vertical or subvertical face for a few weeks, until vegetation becomes established.
5.6.1.4.2 Other Protective Works

More costly types of protection, such as stone pitching (possibly grouted), gabions, masonry or placing of concrete may also be used, but, in general, they are economically justified only where the overall slope stability has to be improved.

5.6.2 Protection of Ditches and Channels

5.6.2.1 Critical Length of Unlined Ditches

The critical length of unlined ditches must be determined, with regard to erosion control. The critical length is defined as the maximum length of unlined ditch, in which water velocities do not give rise to erosion.

The maximum velocity of water can be calculated from the slope, shape and dimensions of the ditch, volume of water and from the roughness coefficient of the material. Knowing the maximum permissible velocity for each type of material, the maximum length of ditch in this material can then be determined. The recommended maximum permissible velocities for different types of material are as follows:

Table 5.1: Maximum velocity of water flow

<table>
<thead>
<tr>
<th>Material</th>
<th>Max. permissible velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine sand</td>
<td>0.3*</td>
</tr>
<tr>
<td>Silt – Coarse sand</td>
<td>0.4 – 0.6*</td>
</tr>
<tr>
<td>Silty Clay – fine gravel</td>
<td>0.5 – 0.8*</td>
</tr>
<tr>
<td>Stiff clay</td>
<td>0.9 – 1.3</td>
</tr>
<tr>
<td>Coarse Gravel</td>
<td>1.2 – 1.7</td>
</tr>
<tr>
<td>Soft rock – Conglomerate</td>
<td>1.8 – 2.5</td>
</tr>
<tr>
<td>Hard rock – Masonry - Concrete</td>
<td>3.0+</td>
</tr>
</tbody>
</table>

*Where the materials are grassed, the maximum permissible velocity is of the order of 1.5 m/s if a good cover is provided and 1.1 m/s if a sparse cover is provided.

5.6.2.2 Methods of Protection

Sections of ditch beyond the critical length must be protected from erosion by lining. The following methods may be used:

- Grassing
- Turfing
- Stone pitching (possibly grouted)
- Placing of masonry
- Concreting
- Reducing the gradient and constructing steps (the steps must be paved)
- Placing velocity breakers
5.6.2.3 SEDIMENTATION CONTROL

If water velocities are too low sedimentation may occur. Ditches and drains should therefore be given sufficient gradient everywhere, in so far as topography and erosion control will permit. Sedimentation velocities for a few types of material are approximately the following:

<table>
<thead>
<tr>
<th>Material</th>
<th>Velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silt</td>
<td>0.08</td>
</tr>
<tr>
<td>Fine sand</td>
<td>0.15</td>
</tr>
<tr>
<td>Coarse sand</td>
<td>0.20</td>
</tr>
<tr>
<td>Fine gravel</td>
<td>0.30</td>
</tr>
<tr>
<td>Gravel</td>
<td>0.65</td>
</tr>
</tbody>
</table>
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

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

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.

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)

<table>
<thead>
<tr>
<th>Test sieve (mm)</th>
<th>Percentage by mass of total aggregate passing test sieve</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Nominal maximum particle size</td>
</tr>
<tr>
<td></td>
<td>37.5 mm</td>
</tr>
<tr>
<td>50</td>
<td>100</td>
</tr>
<tr>
<td>37.5</td>
<td>95 – 100</td>
</tr>
<tr>
<td>28</td>
<td>-</td>
</tr>
<tr>
<td>20</td>
<td>60 – 80</td>
</tr>
<tr>
<td>10</td>
<td>40 – 60</td>
</tr>
<tr>
<td>5</td>
<td>25 - 40</td>
</tr>
<tr>
<td>2.36</td>
<td>15 – 30</td>
</tr>
<tr>
<td>0.425</td>
<td>7 – 19</td>
</tr>
<tr>
<td>0.075 (1)</td>
<td>5 – 12</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Typical Annual Rainfall (mm)</th>
<th>Minimum 10% Fines Values (kN)</th>
<th>Minimum Ratio Wet/Dry Test (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;500</td>
<td>110</td>
<td>75</td>
</tr>
<tr>
<td>&lt;500</td>
<td>110</td>
<td>60</td>
</tr>
</tbody>
</table>
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.

**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)**

<table>
<thead>
<tr>
<th>Test sieve (mm)</th>
<th>Percentage by mass of total aggregate passing test sieve</th>
<th>Nominal maximum particle size</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>37.5 mm</td>
<td>20 mm</td>
</tr>
<tr>
<td>50</td>
<td>100</td>
<td>-</td>
</tr>
<tr>
<td>37.5</td>
<td>80 – 100</td>
<td>100</td>
</tr>
<tr>
<td>20</td>
<td>60 – 80</td>
<td>80 – 100</td>
</tr>
<tr>
<td>10</td>
<td>45 – 65</td>
<td>55 – 80</td>
</tr>
<tr>
<td>5</td>
<td>30 – 50</td>
<td>40 – 60</td>
</tr>
<tr>
<td>2.36</td>
<td>20 – 40</td>
<td>30 – 50</td>
</tr>
<tr>
<td>0.425</td>
<td>10 – 25</td>
<td>12 – 27</td>
</tr>
<tr>
<td>0.075</td>
<td>5 – 15</td>
<td>5 – 15</td>
</tr>
</tbody>
</table>
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:

\[
PM = PI \times (\text{percentage passing the } 0.425 \text{ mm sieve})
\]

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 Somaliland, the low altitude areas of the northern coastal areas (Guban) are dry throughout most of the year. In these low rainfall areas, typically with a mean annual rainfall of less than 200 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 (T1, 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.

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 Somaliland. 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 Somaliland, 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.
**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.

**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 Somaliland, 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)

<table>
<thead>
<tr>
<th>Climate</th>
<th>Typical Annual Rainfall</th>
<th>Liquid Limit</th>
<th>Plasticity Index</th>
<th>Linear Shrinkage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moist tropical and wet tropical</td>
<td>&gt;500mm</td>
<td>&lt;35</td>
<td>&lt;6</td>
<td>&lt;3</td>
</tr>
<tr>
<td>Seasonally wet trop</td>
<td>&gt;500mm</td>
<td>&lt;45</td>
<td>&lt;12</td>
<td>&lt;6</td>
</tr>
<tr>
<td>Arid and semi-arid</td>
<td>&lt;500mm</td>
<td>&lt;55</td>
<td>&lt;20</td>
<td>&lt;10</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Test Sieve (mm)</th>
<th>Percentage by mass of total aggregate passing test sieve (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>100</td>
</tr>
<tr>
<td>37.5</td>
<td>80 – 100</td>
</tr>
<tr>
<td>20</td>
<td>60 – 100</td>
</tr>
<tr>
<td>5</td>
<td>30 – 100</td>
</tr>
<tr>
<td>1.18</td>
<td>17 – 75</td>
</tr>
<tr>
<td>0.3</td>
<td>9 – 50</td>
</tr>
<tr>
<td>0.075</td>
<td>5 – 25</td>
</tr>
</tbody>
</table>

**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 \( \frac{D_{15}(\text{coarse layer})}{D_{85}(\text{fine layer})} \) should be less than 5

where \( D_{15} \) is the sieve size through which 15% by weight of the material passes and \( D_{85} \) is the sieve size through which 85% passes.

b) The ratio \( \frac{D_{50}(\text{coarse layer})}{D_{50}(\text{fine layer})} \) should be less than 25

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

c) The ratio \( \frac{D_{15}(\text{coarse layer})}{D_{15}(\text{fine layer})} \) 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 STABILIZED 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 Somaliland 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

<table>
<thead>
<tr>
<th>Code</th>
<th>Description</th>
<th>Unconfined compressive strength* (Mpa) (Cement Stabilized)</th>
<th>Minimum CBR value* (%) (Lime stabilized)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB1</td>
<td>Stabilized base course</td>
<td>3.0 - 6.0</td>
<td>100</td>
</tr>
<tr>
<td>CB2</td>
<td>Stabilized base course</td>
<td>1.5 - 3.0</td>
<td>80</td>
</tr>
<tr>
<td>CS</td>
<td>Stabilized sub-base</td>
<td>0.75 - 1.5</td>
<td>40</td>
</tr>
</tbody>
</table>

* 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

<table>
<thead>
<tr>
<th>PP ≤60</th>
<th>More than 25% passing the 0.075 mm sieve</th>
<th>Less than 25% passing the 0.075 mm sieve</th>
</tr>
</thead>
<tbody>
<tr>
<td>PI ≤10</td>
<td>10&lt;PI ≤20</td>
<td>PI &gt; 20</td>
</tr>
<tr>
<td>Cement</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Lime</td>
<td>*</td>
<td>Yes</td>
</tr>
<tr>
<td>Lime-Pozzolan</td>
<td>Yes</td>
<td>*</td>
</tr>
</tbody>
</table>

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**

<table>
<thead>
<tr>
<th>Test sieve (mm)</th>
<th>Percentage by mass of total aggregate passing sieve (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>CB1</td>
</tr>
<tr>
<td>53</td>
<td>100</td>
</tr>
<tr>
<td>37.5</td>
<td>85 – 100</td>
</tr>
<tr>
<td>20</td>
<td>60 – 90</td>
</tr>
<tr>
<td>5</td>
<td>30 – 65</td>
</tr>
<tr>
<td>2</td>
<td>20 – 50</td>
</tr>
<tr>
<td>0.425</td>
<td>10 – 30</td>
</tr>
<tr>
<td>0.075</td>
<td>5 – 15</td>
</tr>
<tr>
<td>LL</td>
<td>25</td>
</tr>
<tr>
<td>PI</td>
<td>6</td>
</tr>
<tr>
<td>LS</td>
<td>3</td>
</tr>
</tbody>
</table>

Maximum allowable value

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

**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.

**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

**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.

**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.

**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 surfacing is 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

<table>
<thead>
<tr>
<th>Property</th>
<th>Test</th>
<th>Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cleanliness</td>
<td>Sedimentation or Decantation&lt;sup&gt;1&lt;/sup&gt;</td>
<td>&lt; 5 per cent passing 0.075mm sieve</td>
</tr>
<tr>
<td>Particle shape</td>
<td>Flakiness index&lt;sup&gt;2&lt;/sup&gt;</td>
<td>&lt; 45 per cent</td>
</tr>
<tr>
<td>Strength</td>
<td>Aggregate Crushing Value (ACV)&lt;sup&gt;3&lt;/sup&gt;</td>
<td>&lt; 25. For weaker aggregates the Ten per Cent Fines Value Test (TFV) is used</td>
</tr>
<tr>
<td></td>
<td>Aggregate Impact Value (AIV)&lt;sup&gt;3&lt;/sup&gt;</td>
<td>&lt; 25</td>
</tr>
<tr>
<td></td>
<td>Los Angeles Abrasion Value (LAA)&lt;sup&gt;4&lt;/sup&gt;</td>
<td>&lt; 30 (wearing course) &lt; 35 (other)</td>
</tr>
<tr>
<td>Abrasion</td>
<td>Aggregate Abrasion Value (AAV)&lt;sup&gt;3&lt;/sup&gt;</td>
<td>&lt; 15</td>
</tr>
<tr>
<td></td>
<td>Los Angeles Abrasion Value (LAA)&lt;sup&gt;4&lt;/sup&gt;</td>
<td>&lt; 12 (very heavy traffic)</td>
</tr>
<tr>
<td>Polishing</td>
<td>Polished Stone Value&lt;sup&gt;3&lt;/sup&gt;</td>
<td>Not less than 50-75 depending on location</td>
</tr>
<tr>
<td>Polishing</td>
<td>Soundness:&lt;sup&gt;5&lt;/sup&gt;</td>
<td>&lt; 12 per cent</td>
</tr>
<tr>
<td></td>
<td>Sodium Test</td>
<td>&lt;___ 18 per cent</td>
</tr>
<tr>
<td></td>
<td>Magnesium Test</td>
<td>Magnesium Test &lt; 18 per cent</td>
</tr>
<tr>
<td>Polishing</td>
<td>Water Absorption&lt;sup&gt;6&lt;/sup&gt;</td>
<td>&lt; 2 per cent</td>
</tr>
<tr>
<td>Durability</td>
<td>Coating and Stripping&lt;sup&gt;7&lt;/sup&gt;</td>
<td>Non-stripped area of aggregate &gt; 95 per cent</td>
</tr>
<tr>
<td></td>
<td>Immersion Tray Test (7)</td>
<td>Effect of water on Index of retained cohesion of compacted stability &gt; 75 per cent</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

To perform satisfactorily as road surfacing, 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.
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.

**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

<table>
<thead>
<tr>
<th>Mix designation</th>
<th>WC1</th>
<th>WC2</th>
<th>BC1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test sieve (mm)</td>
<td>Wearing course</td>
<td>Binder Course</td>
<td>Percentage by mass of total aggregate passing test sieve</td>
</tr>
<tr>
<td>28</td>
<td>28 – 72</td>
<td>28 – 72</td>
<td>100</td>
</tr>
<tr>
<td>20</td>
<td>100</td>
<td>-</td>
<td>80 – 100</td>
</tr>
<tr>
<td>10</td>
<td>60 – 80</td>
<td>60 – 80</td>
<td>60 – 80</td>
</tr>
<tr>
<td>5</td>
<td>54 – 72</td>
<td>62 – 80</td>
<td>36 – 56</td>
</tr>
<tr>
<td>2.36</td>
<td>42 – 58</td>
<td>44 – 60</td>
<td>28 – 44</td>
</tr>
<tr>
<td>1.18</td>
<td>34 – 48</td>
<td>36 – 50</td>
<td>20 – 37</td>
</tr>
<tr>
<td>0.6</td>
<td>26 – 38</td>
<td>28 – 40</td>
<td>15 – 27</td>
</tr>
<tr>
<td>0.3</td>
<td>18 – 28</td>
<td>20 – 30</td>
<td>10 – 20</td>
</tr>
<tr>
<td>0.15</td>
<td>12 – 20</td>
<td>12 – 20</td>
<td>5 – 13</td>
</tr>
<tr>
<td>0.075</td>
<td>6 – 12</td>
<td>6 – 12</td>
<td>2 – 6</td>
</tr>
</tbody>
</table>
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**

<table>
<thead>
<tr>
<th>Total Traffic (10^6 ESA)</th>
<th>&lt; 1.5</th>
<th>1.5 - 10.0</th>
<th>&gt; 10.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traffic classes</td>
<td>T1,T2,T3</td>
<td>T4,T5,T6</td>
<td>T7,T8</td>
</tr>
<tr>
<td>Minimum stability (kN at 60°C)</td>
<td>3.5</td>
<td>6.0</td>
<td>7.0</td>
</tr>
<tr>
<td>Minimum flow (mm)</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Compaction level (Number of blows)</td>
<td>2 x 50</td>
<td>2 x 75</td>
<td>2 x 75</td>
</tr>
<tr>
<td>Air voids (per cent)</td>
<td>3 - 5</td>
<td>3 - 5</td>
<td>3 – 5</td>
</tr>
</tbody>
</table>

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**

<table>
<thead>
<tr>
<th>Nominal maximum particle size (mm)</th>
<th>Minimum voids in mineral aggregate (per cent)</th>
</tr>
</thead>
<tbody>
<tr>
<td>37.5</td>
<td>12</td>
</tr>
<tr>
<td>28</td>
<td>12.5</td>
</tr>
<tr>
<td>20</td>
<td>14</td>
</tr>
<tr>
<td>144</td>
<td>15</td>
</tr>
<tr>
<td>10</td>
<td>16</td>
</tr>
<tr>
<td>5</td>
<td>18</td>
</tr>
</tbody>
</table>

**BITUMEN MACADAM**

Close graded bitumen macadam is 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 both wearing course and base course mixes are are given in Table 8-6. Sealing the wearing course with a surface dressing soon after laying is recommended for a long maintenance-free life. Slurry seals can also be used but they are best used in combination with a surface dressing to form a Cape seal.

Table 8-6: Bitumen Macadam surfacings

<table>
<thead>
<tr>
<th>Mix designation</th>
<th>BC2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Binder Course</strong></td>
<td></td>
</tr>
<tr>
<td>Test sieve (mm)</td>
<td>Percentage by mass of total aggregate passing test sieve</td>
</tr>
<tr>
<td>28</td>
<td>100</td>
</tr>
<tr>
<td>20</td>
<td>95 – 100</td>
</tr>
<tr>
<td>14</td>
<td>65 – 85</td>
</tr>
<tr>
<td>10</td>
<td>52 – 72</td>
</tr>
<tr>
<td>6.3</td>
<td>39 – 55</td>
</tr>
<tr>
<td>3.35</td>
<td>32 – 46</td>
</tr>
<tr>
<td>1.18</td>
<td>-</td>
</tr>
<tr>
<td>0.3</td>
<td>7 – 21</td>
</tr>
<tr>
<td>0.075 (1)</td>
<td>2 – 8</td>
</tr>
</tbody>
</table>

Bitumen content (2) (percent 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

<table>
<thead>
<tr>
<th>Design Traffic</th>
<th>&lt; 1.5</th>
<th>1.5 - 10.0</th>
<th>&gt; 10.0</th>
<th>Severe Sites</th>
</tr>
</thead>
</table>

Page 8-6

Roads Development Agency
8.3.3 **Rolled asphalt**

Rolled asphalt is a gap-graded mix which relies for its properties primarily on the mortar of bitumen, filler (<0.075 mm) and fine aggregate (0.075–2.36 mm). The coarse aggregate (>2.36 mm) acts as an extender but its influence on stability and density increases as the proportion of coarse aggregate in the mix increases above approximately 55 per cent. If the coarse aggregate content is less than about 40 per cent, pre-coated chippings should be rolled into the surface to provide texture for good skid resistance where necessary.

It is generally recognized that these mixtures are more durable and flexible than the continuously graded asphaltic concretes. However, they can be more sensitive to deformation under heavy traffic in hot weather and careful design of the mixture is necessary to avoid this risk.

Also, rolled asphalt mixtures can be used in surfacing roads over the weak subgrades of Ethiopia (such as black cotton soils) to take advantage of their flexibility characteristics.

Rolled asphalt has been developed elsewhere to recipe specifications but can also be designed using the Marshall Test so that the physical characteristics of the fine aggregate can be taken into account (British Standard 594). Wearing courses made to the particle size distributions in the British Standard and with filler-to-binder ratios in the range 0.8–1.0 have performed well in the tropics. The compositions of suitable mixes are summarized in Table 8.7. The mixes made with natural sand are more tolerant of proportioning errors than asphaltic concrete and are easier to compact. Although the air voids tend to be slightly higher than asphaltic concrete, they are discontinuous and the mixes are impermeable.

**TABLE 8.7**
Rolled asphalt surfacings

<table>
<thead>
<tr>
<th>Mix designation</th>
<th>WC5</th>
<th>WC6</th>
<th>BC3</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Wearing Course (1,2)</strong></td>
<td>Binder Course</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Test sieve (mm)</td>
<td>Percentage by mass of total aggregate passing test sieve</td>
<td></td>
<td></td>
</tr>
<tr>
<td>28</td>
<td>100</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Type of fines

<table>
<thead>
<tr>
<th>Natural sand</th>
<th>Crushed rock</th>
<th>Sand or crushed rock</th>
</tr>
</thead>
</table>

### Bitumen grade (pen)

<table>
<thead>
<tr>
<th>40/50 or 60/70</th>
<th>60/70</th>
<th>40/50 or 60/70</th>
</tr>
</thead>
</table>

### Thickness (mm)

<table>
<thead>
<tr>
<th>50</th>
<th>50</th>
<th>40-65</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>80</td>
<td></td>
</tr>
</tbody>
</table>

### Bitumen content (per cent)

<table>
<thead>
<tr>
<th>Minimum target value</th>
<th>6.5 ± 0.6</th>
</tr>
</thead>
<tbody>
<tr>
<td>(crushed rock)</td>
<td></td>
</tr>
<tr>
<td>by mass of total mix</td>
<td>6.3 ± 0.5 (3)</td>
</tr>
<tr>
<td>(gravel)</td>
<td>6.3 ± 0.6</td>
</tr>
</tbody>
</table>

Notes:
1. The preferred target for coarse aggregate is 50 per cent.
2. For WC5 a maximum of 12 per cent should be retained between the 0.6 mm and 2.36 mm sieves.
3. With 50% coarse aggregate (see BS594).

**Flexible 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.

**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.

**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>
<thead>
<tr>
<th>Mix designation</th>
<th>RB1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test sieve (mm)</td>
<td>Percentage by mass of total aggregate passing test sieve</td>
</tr>
<tr>
<td>50</td>
<td>100</td>
</tr>
</tbody>
</table>
### Bitumen-Bound Materials

<table>
<thead>
<tr>
<th>Material</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>37.5 – 95</td>
</tr>
<tr>
<td>Gravel</td>
<td>28 – 70</td>
</tr>
<tr>
<td>Gravel</td>
<td>14 – 56</td>
</tr>
<tr>
<td>Gravel</td>
<td>10 – 32</td>
</tr>
<tr>
<td>Gravel</td>
<td>5 – 44</td>
</tr>
<tr>
<td>Gravel</td>
<td>0.3 – 7</td>
</tr>
<tr>
<td>0.075</td>
<td>0.075 – 2</td>
</tr>
</tbody>
</table>

### Bitumen Grades
- 37.5 – 95
- 28 – 70
- 14 – 56
- 10 – 32
- 5 – 44
- 0.3 – 7
- 0.075

### 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:

1. **Trials for mix production, laying and compaction should be carried out to determine suitable mix proportions and procedures.**
2. **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.**
3. **Mixing times and temperatures should be set at the minimum required to achieve good coating of the aggregates and satisfactory compaction.**
4. **The highest bitumen content commensurate with adequate stability should be used.**

### 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**

<table>
<thead>
<tr>
<th>Traffic Classes</th>
<th>Traffic Classes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Marshall stability at 60°C (min)</td>
<td>1 kN</td>
</tr>
<tr>
<td>Marshall flow value at 60°C (max)</td>
<td>2.5 mm</td>
</tr>
</tbody>
</table>

### 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**

<table>
<thead>
<tr>
<th>Combined aggregate passing test sieve (mm)</th>
<th>Bitumen content</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test sieve</td>
<td>Per cent</td>
</tr>
<tr>
<td>12.5+</td>
<td>± 5</td>
</tr>
<tr>
<td>10.0</td>
<td>± 5</td>
</tr>
<tr>
<td>2.36</td>
<td>± 5</td>
</tr>
</tbody>
</table>
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 subgrade.

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.

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.

![Diagram of surface treatments](image)

**Figure 9-1: Types of Surface Treatments**

**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 if traffic is allowed to run on the first treatment for a minimum period of 2-3 weeks (and preferable longer) before the second treatment is applied. This allows the chippings of the first treatment to adopt a stable interlocking mosaic, which provides a firm foundation for the second treatment. However, traffic and animals may cause contamination of the surface with mud or soil during this period and this must be thoroughly swept off before the second treatment is applied. Such cleaning is sometimes difficult to achieve and the early application of the second seal to prevent such contamination may give a better result. Sand may sometimes be used as an alternative to chippings for the second treatment. Although it cannot contribute to the overall all thickness of the surfacing, the combination of binder and sand provides a useful grouting medium for the chipping of the first seal and helps to hold them in place more firmly when they are poorly shaped. A slurry seal may also be used for the same purpose (see below).

**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.

**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.

**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**

<table>
<thead>
<tr>
<th>Grading Limits</th>
<th>Nominal Size of Aggregates (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Sieve</td>
<td>20</td>
</tr>
<tr>
<td>28</td>
<td>100</td>
</tr>
<tr>
<td>20</td>
<td>85-100</td>
</tr>
<tr>
<td>14</td>
<td>0-35</td>
</tr>
<tr>
<td>10</td>
<td>0-7</td>
</tr>
<tr>
<td>6.3</td>
<td>-</td>
</tr>
<tr>
<td>5.0</td>
<td>-</td>
</tr>
<tr>
<td>3.35</td>
<td>-</td>
</tr>
<tr>
<td>2.36</td>
<td>0-2</td>
</tr>
<tr>
<td>0.600</td>
<td>-</td>
</tr>
<tr>
<td>0.075</td>
<td>0-1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specified Size</th>
<th>Minimum Percentage by Mass Retained on Test Sieve</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>65</td>
</tr>
<tr>
<td>Maximum Flakiness Index</td>
<td>25</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Grading Limits</th>
<th>Nominal Size of Aggregates (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Sieve</td>
<td>20</td>
</tr>
<tr>
<td>28</td>
<td>100</td>
</tr>
<tr>
<td>20</td>
<td>85-100</td>
</tr>
<tr>
<td>14</td>
<td>0-40</td>
</tr>
<tr>
<td>10</td>
<td>0-7</td>
</tr>
<tr>
<td>6.3</td>
<td>-</td>
</tr>
<tr>
<td>5.0</td>
<td>-</td>
</tr>
<tr>
<td>3.35</td>
<td>-</td>
</tr>
<tr>
<td>2.36</td>
<td>0-3</td>
</tr>
<tr>
<td>0.600</td>
<td>0-2</td>
</tr>
<tr>
<td>0.075</td>
<td>-</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specified Size</th>
<th>Minimum Percentage by Mass Retained on Test Sieve</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>60</td>
</tr>
<tr>
<td>Maximum Flakiness Index</td>
<td>35</td>
</tr>
</tbody>
</table>
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>
<thead>
<tr>
<th>Site Definition</th>
<th>Traffic (cv/l/d) at Design Life</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0 to 100</td>
</tr>
<tr>
<td>1 Dual carriageway non-event sections and minor junctions</td>
<td>55</td>
</tr>
<tr>
<td>2 Single carriageway non-event sections and minor junctions</td>
<td>45</td>
</tr>
<tr>
<td>3 Approaches to and across major junctions (all limbs)</td>
<td></td>
</tr>
<tr>
<td>Gradient 5%-10%, Longer than 50m</td>
<td>50</td>
</tr>
<tr>
<td>Bend, radius 100-250m.</td>
<td></td>
</tr>
<tr>
<td>Roundabout</td>
<td></td>
</tr>
<tr>
<td>4 Gradient &gt; 10%, longer than 50m</td>
<td>55</td>
</tr>
<tr>
<td>Bend, radius &lt; 100m</td>
<td></td>
</tr>
<tr>
<td>5 Approach to roundabout, traffic signals, pedestrian crossing, railway level crossing, etc.</td>
<td>63</td>
</tr>
</tbody>
</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 if proper care and attention is not given to all other aspects of the surface treatment process.

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

- Spraying the chippings with a light application of creosote, diesel oil, or kerosene at ambient temperature (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°C for bitumen binders and 120°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

<table>
<thead>
<tr>
<th>Nominal Size of Chippings (mm)</th>
<th>Target Binder Content (per cent by mass)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bitumen</td>
</tr>
<tr>
<td>6</td>
<td>1.0</td>
</tr>
<tr>
<td>10</td>
<td>0.8</td>
</tr>
<tr>
<td>14</td>
<td>0.6</td>
</tr>
<tr>
<td>20</td>
<td>0.5</td>
</tr>
</tbody>
</table>

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.

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/m². 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

<table>
<thead>
<tr>
<th>Grade of Cutback Binder</th>
<th>Permitted Viscosity Range (Centistokes at 60°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MC 250</td>
<td>250-500</td>
</tr>
<tr>
<td>MC 70</td>
<td>70-140</td>
</tr>
<tr>
<td>MC 30</td>
<td>30-60</td>
</tr>
</tbody>
</table>

**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 Somaliland, 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
**PENETRATION GRADE BITUMENS**

Penetration grade bitumens vary between 80/100 to approximately 700 penetration. The softer penetration grade binders are usually produced at the refinery but can be made in the field by blending appropriate amounts of kerosene, diesel, or a blend of kerosene and diesel. With higher solvent contents the binder has too low a viscosity to be classed as being of penetration grade and is then referred to as 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°C for long periods of the year.

**BITUMEN EMULSION**

Cationic bitumen emulsion with a bitumen content of 70 to 75 per cent is recommended for most surface treatment work. This type of binder can be applied through whirling spray jets at a temperature between 70 and 85°C 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.

**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 Somaliland, 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 Somaliland. 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 Somaliland (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](image)

![Figure 9-4: Viscosity/Temperature Relationships for Blends of 80/100 Pen Bitumen with Diesel Fuel](image)
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.

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.

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 underlying 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)

<table>
<thead>
<tr>
<th>Type of Surface</th>
<th>2000-4000</th>
<th>1000-2000</th>
<th>200-1000</th>
<th>20-200</th>
<th>Less Than 20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Hard</td>
<td>10</td>
<td>10</td>
<td>6</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>Hard</td>
<td>14</td>
<td>14</td>
<td>10</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>Normal</td>
<td>20</td>
<td>14</td>
<td>10</td>
<td>10</td>
<td>6</td>
</tr>
<tr>
<td>Soft</td>
<td>*</td>
<td>20</td>
<td>14</td>
<td>14</td>
<td>10</td>
</tr>
<tr>
<td>Very Soft</td>
<td>*</td>
<td>*</td>
<td>20</td>
<td>14</td>
<td>10</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Category of Surface</th>
<th>Penetration at 30°C</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Hard</td>
<td>0-2</td>
<td>Concrete or very lean bituminous structures with dry stony surfaces. There would be negligible penetration of chippings under the heaviest traffic.</td>
</tr>
<tr>
<td>Hard</td>
<td>2-5</td>
<td>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.</td>
</tr>
<tr>
<td>Normal</td>
<td>5-8</td>
<td>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.</td>
</tr>
<tr>
<td>Soft</td>
<td>8-12</td>
<td>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.</td>
</tr>
<tr>
<td>Very Soft</td>
<td>&gt;12</td>
<td>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.</td>
</tr>
</tbody>
</table>

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 or 14mm 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.
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 $7 \times 10^5$ 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.

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°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 use 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.

**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 Somaliland.

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.
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
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

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

### 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.

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

Where:
- \( R \) = Basic rate of spread of bitumen (kg/m²)
- \( F \) = Overall weighting factor
- \( ALD \) = The average least dimension of the chippings (mm)

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 liters/m²; however, calibration of a distributor is easier to do by measuring spray rates in terms of mass.

### 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.
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).

![Surface Treatment Design Chart](image)

**Figure 9-7 Surface Treatment Design Chart**
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

<table>
<thead>
<tr>
<th>Binder Grade</th>
<th>Basic Spray Rate from Figure 9-7 or Equation 1</th>
<th>Flat Terrain, Moderate Traffic Speed</th>
<th>High Speed Traffic, Down-Hill Grades &gt;3%</th>
<th>Low Speed Traffic, Up-Hill Grades &gt;3%</th>
</tr>
</thead>
<tbody>
<tr>
<td>MC 3000 300 pen</td>
<td>R</td>
<td>R</td>
<td>R*1.1</td>
<td>R*0.9</td>
</tr>
<tr>
<td>80/100 pen Emulsion¹</td>
<td>R</td>
<td>R*0.95</td>
<td>R*1.05</td>
<td>R*0.86</td>
</tr>
<tr>
<td></td>
<td>R</td>
<td>R*(90/%) binder</td>
<td>R*0.99</td>
<td>R*0.81</td>
</tr>
<tr>
<td></td>
<td>R</td>
<td>R*(99/%) binder</td>
<td>R*(99/%) binder</td>
<td>R*(81/%) binder</td>
</tr>
</tbody>
</table>

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.

**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.

**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

<table>
<thead>
<tr>
<th>Category of Road Surface Hardness</th>
<th>Nominal Chipping Size (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very hard</td>
<td>6</td>
</tr>
<tr>
<td>Hard</td>
<td>6</td>
</tr>
<tr>
<td>Normal</td>
<td>10</td>
</tr>
<tr>
<td>Soft</td>
<td>14</td>
</tr>
</tbody>
</table>

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.

**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:

$$ \text{Chipping application rate (kg/m}^2\) = 1.364 \times \text{ALD} \quad (2) $$
Table 9-11: Suggested Maximum Increases in Bitumen Spray Rate for Low Volume Roads

<table>
<thead>
<tr>
<th>ALD of chippings (mm)</th>
<th>3</th>
<th>6</th>
<th>&gt;6</th>
</tr>
</thead>
<tbody>
<tr>
<td>All traffic (vehicles/lane/day)</td>
<td>&lt;20</td>
<td>20-100</td>
<td>&lt;20</td>
</tr>
<tr>
<td>Increase in bitumen spray rate (per cent)</td>
<td>15</td>
<td>10</td>
<td>20</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Traffic (Heavy)</th>
<th>-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Existing Surface (Average Bituminous)</td>
<td>-1</td>
</tr>
<tr>
<td>Chippings (Cubical)</td>
<td>0</td>
</tr>
<tr>
<td>Climate (Hot/Dry)</td>
<td>-1</td>
</tr>
<tr>
<td>Overall Weighing Factor (F)</td>
<td>-5</td>
</tr>
</tbody>
</table>

Aggregate (Nominal 19 mm)

<table>
<thead>
<tr>
<th>Medium Size (i.e. 50 per cent passing)</th>
<th>16mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flakiness Index</td>
<td>16</td>
</tr>
<tr>
<td>Average Least Dimension (from Nomograph, Figure 9-6)</td>
<td>12</td>
</tr>
</tbody>
</table>
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

<table>
<thead>
<tr>
<th>Type of Terrain</th>
<th>Basic Spread Rate R for MC 3000 (from Fig. 9-7 or Equation 1) (kg/m²)</th>
<th>For Increased Durability R_D = (R * 1.08) (kg/m²)</th>
<th>Spread Rates for Penetration Grade Binders (kg/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flat</td>
<td>0.89</td>
<td>0.96</td>
<td>0.87 0.92</td>
</tr>
<tr>
<td>Uphill Grade &gt; 3%</td>
<td>0.89*0.9 = 0.80</td>
<td>0.87</td>
<td>0.78 0.84</td>
</tr>
<tr>
<td>Downhill Grade &gt; 3%</td>
<td>0.89*1.1 = 0.98</td>
<td>1.06</td>
<td>0.95 1.02</td>
</tr>
</tbody>
</table>

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.

**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

<table>
<thead>
<tr>
<th>BS Test Sieve (mm)</th>
<th>Fine</th>
<th>General</th>
<th>Coarse</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>-</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>5.0</td>
<td>100</td>
<td>90-100</td>
<td>70-90</td>
</tr>
<tr>
<td>2.36</td>
<td>65-90</td>
<td>50</td>
<td>45-70</td>
</tr>
<tr>
<td>1.18</td>
<td>40-60</td>
<td>18-30</td>
<td>28-50</td>
</tr>
<tr>
<td>0.6</td>
<td>25-42</td>
<td>10-21</td>
<td>12-25</td>
</tr>
<tr>
<td>0.3</td>
<td>15-30</td>
<td>5-15</td>
<td>7-18</td>
</tr>
<tr>
<td>0.15</td>
<td>10-20</td>
<td>5-15</td>
<td>5-15</td>
</tr>
<tr>
<td>0.075</td>
<td>10-16</td>
<td>7.5-13.5</td>
<td>6.5-12.0</td>
</tr>
</tbody>
</table>

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’

<table>
<thead>
<tr>
<th>Size of Chipping in Surface treatment (mm)</th>
<th>Coverage (m²/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>130-170</td>
</tr>
<tr>
<td>14</td>
<td>170-240</td>
</tr>
<tr>
<td>10</td>
<td>180-250</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Sieve (mm)</th>
<th>Dense</th>
<th>Coarse</th>
</tr>
</thead>
<tbody>
<tr>
<td>19</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>16</td>
<td>79-100</td>
<td>77-100</td>
</tr>
<tr>
<td>12</td>
<td>61-100</td>
<td>59-100</td>
</tr>
<tr>
<td>9.5</td>
<td>42-100</td>
<td>40-85</td>
</tr>
<tr>
<td>4.75</td>
<td>19-68</td>
<td>17-46</td>
</tr>
<tr>
<td>2.36</td>
<td>8-51</td>
<td>1-20</td>
</tr>
<tr>
<td>1.18</td>
<td>6-40</td>
<td>0-10</td>
</tr>
<tr>
<td>0.60</td>
<td>3-30</td>
<td>0-3</td>
</tr>
<tr>
<td>0.30</td>
<td>2-21</td>
<td>0-2</td>
</tr>
<tr>
<td>0.15</td>
<td>1-16</td>
<td>0-1</td>
</tr>
<tr>
<td>0.075</td>
<td>0-10</td>
<td>0-1</td>
</tr>
</tbody>
</table>

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 l/m² 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 three 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.

**SAND 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 m³/m² (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/m² depending on the type of surface being sealed.

**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.

**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/m². 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/m² 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>
<thead>
<tr>
<th>CHART NO</th>
<th>SURFACING</th>
<th>BASE COURSE</th>
<th>REFER TO CHAPTERS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Double surface dressing</td>
<td>T1 - T4 use GB1, GB2 or GB3</td>
<td>6 and 9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>T5 use GB1 or GB2</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>T6 must be GB1</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Double surface dressing</td>
<td>T1 - T4 use GB1, GB2 or GB3</td>
<td>6, 7 and 8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>T5 use GB1 or GB2</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>T6 - T8 must be GB1</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>“Flexible” asphalt</td>
<td>T1 - T5 use GB1 or GB2</td>
<td>6 and 8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>T6 use GB1</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>“Flexible” asphalt</td>
<td>T1 - T5 use GB1 or GB2</td>
<td>6, 7 and 8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>T6 - T8 use GB1</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Wearing course and Base course</td>
<td>GB1</td>
<td>6 and 8</td>
</tr>
<tr>
<td>6</td>
<td>Wearing course and Base course</td>
<td>GB1 or GB2</td>
<td>6, 7 and 8</td>
</tr>
<tr>
<td>7</td>
<td>High quality single seal or</td>
<td>RB1, RB2 or RB3</td>
<td>8 and 9</td>
</tr>
<tr>
<td></td>
<td>double seal for T4</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>“Flexible” asphalt for T5-T8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Double surface dressing</td>
<td>CB1, CB2</td>
<td>7 and 9</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Design Chart No.</th>
<th>No. 4</th>
<th>No. 5</th>
<th>No. 6</th>
<th>No. 7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pave. Comps. &amp; Sel. Fill</td>
<td>Pavement Structures</td>
<td>Alternate Structure No. 1</td>
<td>Alternate Structure No. 2</td>
<td>Alternate Structure No. 3</td>
</tr>
<tr>
<td>Surfacing (asphalt concrete) (1)</td>
<td>5 cm AC</td>
<td>15 cm AC</td>
<td>15 cm AC</td>
<td>5 cm AC</td>
</tr>
<tr>
<td>Roadbase:</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>· Crushed Stone</td>
<td>15 cm</td>
<td>25 cm</td>
<td>15 cm</td>
<td>—</td>
</tr>
<tr>
<td>· Cement stabilized (e.g. 4 Mpa)</td>
<td>15 cm</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>· Cement stabilized (e.g. 2.5 Mpa)</td>
<td>12.5 cm</td>
<td>—</td>
<td>22.5 cm</td>
<td>—</td>
</tr>
<tr>
<td>· Bituminous stabilized</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>20 cm</td>
</tr>
<tr>
<td>Granular subbase</td>
<td>—</td>
<td>27.5 cm</td>
<td>—</td>
<td>27.5 cm (2)</td>
</tr>
<tr>
<td>Selected fill</td>
<td>15 cm</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>RELATIVE UNIT COST</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt Concrete</td>
<td></td>
</tr>
<tr>
<td>5 cm thick</td>
<td>0.33</td>
</tr>
<tr>
<td>15 cm thick*</td>
<td>0.87</td>
</tr>
<tr>
<td>Bituminous stabilized roadbase</td>
<td></td>
</tr>
<tr>
<td>20 cm thick</td>
<td>1.00</td>
</tr>
<tr>
<td>Crushed stone roadway</td>
<td></td>
</tr>
<tr>
<td>15 cm thick</td>
<td>0.56</td>
</tr>
<tr>
<td>25 cm thick</td>
<td>0.90</td>
</tr>
<tr>
<td>Cement stabilized roadbase, 4 MPa</td>
<td></td>
</tr>
<tr>
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*wearign course and binder course

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

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<td>+27.5 cm granular subbase</td>
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<td>+22.5 cm cement stabilized roadbase (2.5 Mpa)</td>
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<td>5 cm AC</td>
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## Key to Structural Catalogue

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### Material Definitions

- **Double surface dressing**
- **Flexible bituminous surface**
- **Bituminous surface** (Usually a wearing course, WC, and a base course, 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**

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**Figure 10-1: Key to Structural Catalog**
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<th>Chart 1</th>
<th>Granular Roadbase / Surface Dressing</th>
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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.
2. The substitution ratio of sub-base to selected fill is 25mm : 32mm.
3. A cement or lime-stabilised sub-base may also be used.
### Chart 2: Composite Road Base (Unbound & Cemented) / Surface Dressing

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Note: Sub-base to fill substitution not permitted.
### CHART 3  GRANULAR ROADBASE / SEMI-STRUCTURAL SURFACE

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**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.

2. The substitution ratio of sub-base to selected fill is 25mm : 32mm.

3. A cement or lime-stabilized sub-base may also be used.
### Chart 4: Composite Roadbase / Semi - Structural Surface

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Note: Sub-base to fill substitution not permitted.
### Chart 5: Granular Roadbase / Structural Surface

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**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.
2. The substitution ratio of sub-base to selected fill is 25mm : 32mm.
3. A cement or lime-stabilised sub-base may also be used.
## Chart 6: Composite Roadbase / Structural Surface

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*Note: Sub-base to fill substitution not permitted.*
### Chart 7  Bituminous Roadbase / Semi-Structural Surface

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**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

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*Note: A granular sub-base may also be used.*
11 PROBLEMATIC SOILS

This chapter describes particular subgrade soils which can be a problem to the highway engineer in road construction. They are described as ‘Low strength soils’, ‘Expansive soils’, ‘Saline soils’ and ‘Organic’ soils.

11.1 LOW STRENGTH SOILS

Soils with CBR < 3%, or < 2% in arid areas, occurring within the design depth, are defined as ‘Low Strength Soils’. Before they can be included in the foundation structure they require treatment which can include one or several of the following measures:

- Removal and replacement
- Chemical stabilisation with either lime and/or cement
- Mechanical stabilisation, or
- Raising of the vertical alignment to increase cover, thereby re-defining the design depth

Details regarding the treatment of such soils will vary according to soil properties, site conditions, available equipment, alternative materials and parallel experience and will be determined at the time of the project.

11.2 EXPANSIVE SOILS

11.2.1 Definition

Otherwise known as ‘black cotton soil’ because of its characteristic appearance, the main property of expansive soil is the significant volume changes it undergoes when wetted and dried. When a road is sealed a strip of land is created under the road which is protected from seasonal variation of rainfall. The centre of the strip will be subject to different moisture (and therefore volume) changes. This can result in longitudinal cracking along the road edges if it is founded on black cotton soil, which become more accentuated with time and progressively extend towards the center of the road.

Figure 11.1: Moisture Variation in Expansive Soils
11.2.2 Distribution
It is believed that the primary source of residual expansive clay soils is the in situ weathering of basic igneous, metamorphic and pyroclastic rocks, which occur in Somaliland. Thus, expansive soil is quite common and local knowledge is very useful to identify areas where it can be a problem. Typically it forms in flat, poorly drained environments which favour the formation of the expansive soil minerals.

11.2.3 Identification
Apart from their appearance other indicators of expansive soils are described in Table 11.1:

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Typical Features of Expansive Soils</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil Type</td>
<td>The more clayey, the more likely to be expansive</td>
</tr>
<tr>
<td>Consistency, when slightly moist to dry</td>
<td>Stiff or very stiff</td>
</tr>
<tr>
<td>Consistency, when wet</td>
<td>Soft and sticky</td>
</tr>
<tr>
<td>Structure</td>
<td>Cracked surface and slickensided fissures</td>
</tr>
<tr>
<td>Colour</td>
<td>Usually dark but this is not always so</td>
</tr>
</tbody>
</table>

The shrinking and swelling property is caused by the preponderance of the clay mineral montmorillonite. There is no quantitative test to determine the amount of montmorillonite present but normal classification tests enable the severity of the expansiveness to be established, as explained below.

It has been found that the ratio of Plasticity Index to clay fraction is more or less constant for any one soil, but this constant varies depending on clay type. The correlation between PI and clay type is termed ‘Activity’, where:

Activity = PI/clay fraction

To be consistent, the clay fraction is expressed as that portion of the soil sample passing the 0.425mm sieve, rather than the percentage passing 2µm. On this basis clays can be classified into four groups as shown in Table 11.2.

<table>
<thead>
<tr>
<th>Description</th>
<th>Activity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inactive clays</td>
<td>&lt;0.75</td>
</tr>
<tr>
<td>Normal clays</td>
<td>0.75 to 1.25</td>
</tr>
<tr>
<td>Active clays</td>
<td>1.25 to 2</td>
</tr>
<tr>
<td>Highly active clays</td>
<td>&gt;2</td>
</tr>
</tbody>
</table>

If the presence of active or highly active clays is established, further testing involving the determination of the Shrinkage Limit (KS 999 Part 2: 2001) is advisable. The Expansiveness, $e_{ex}$, is calculated from the following empirical formula:

$$e_{ex} = 2.4 \times Wp - 3.9 \times Ws + 32.5$$
where \( W_p = \) Plastic Limit * fraction passing the 425\( \mu \)m sieve/100
and \( W_s = \) Shrinkage Limit * fraction passing the 425\( \mu \)m sieve/100

Expansiveness is then classified according to Table 11.2:

**Table 11.3: Degree of Expansiveness of Expansive Soils**

<table>
<thead>
<tr>
<th>Expansiveness, ( \varepsilon_{ex} )</th>
<th>Classification</th>
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<tr>
<td>&lt;20</td>
<td>Low</td>
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<tr>
<td>20 to 50</td>
<td>Medium</td>
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<tr>
<td>&gt;50</td>
<td>High</td>
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</tbody>
</table>

11.2.4 Remediation

Four possible treatments are possible to overcome the problem of expansive soils:

- avoid by re-alignment
- excavate and replace with non-expansive materials
- stabilise with lime, or
- minimise moisture changes by engineering measures

11.2.4.1 Re-alignment
This is only possible if the expansive soil is limited in extent.

11.2.4.2 Replacement
This is the simplest and most effective treatment but the cost and effect on the environment by sidecasting large quantities of material must be assessed. In practice it is sufficient to remove the expansive soil to a depth of 1m. Even if some expansive clay remains it should be adequately confined and protected from moisture changes. The backfill should be at least of S2 quality and impermeable enough not to act as a drain.

Embankments should be constructed with suitable fill material as discussed later.

11.2.4.3 Stabilisation
If proper mixing can be achieved, treatment of expansive soil with 4% to 6% of hydrated lime is usually effective and provides the following improvements:

- reduces the Plasticity Index to less than 20
- increases considerably the Shrinkage Limit
- reduces Swell to negligible values, and
- increases the CBR to minimum of 10 (after 7 days cure) and 15 (after 28 days cure)
- alters the grading by agglomeration of the clay particles similar to that of a silt
All these improvements render the treated soil easily workable and it can be assumed that it will become of S4 quality. However, it is costly because a substantial thickness must be treated (minimum 300mm) and therefore is advantageous where no suitable backfill or improved subgrade material exists, or there are strong environmental objections to side casting large amounts of expansive soil.

The mixing procedure is to add the lime in two or three increments, followed by intense mixing by pulvimixer. The mixed soil should be left fallow for two to three days between each mixing operation to enable the lime to take effect. Wet weather makes initiating this operation impossible owing to the physical nature of the soil.

11.2.4.4 Engineering Measures for Construction on Expansive Soils

If none of the above measures can be avoided, special precautions are required to avoid damage to the road structure caused by detrimental volume changes when building on or with expansive soils.

Widening of shoulders is beneficial whenever economically feasible. The zone of seasonal moisture (and volume) change is thus moved further away from the carriageway.

Side drains, if required, should be placed at a minimum distance of 4 to 6 m, depending on road category. Side fill consisting of expansive soil requires protection from erosion by grasses but no trees should be planted or allowed to root on the embankment slope.

Table 11.4 proposes alternative methods of construction over expansive soil:

<table>
<thead>
<tr>
<th>Expansiveness</th>
<th>Paved Trunk Roads</th>
<th>Other Paved Roads</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low $\varepsilon&lt;20$</td>
<td>Sealed shoulders</td>
<td>Side slopes 1:6*</td>
</tr>
<tr>
<td>Medium $\varepsilon_{ex}20-50$</td>
<td>See Fig 11.2</td>
<td>Sealed shoulders</td>
</tr>
<tr>
<td></td>
<td>Earthworks cover min 1m</td>
<td>Earthworks cover min 0.6m</td>
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<tr>
<td>High $\varepsilon_{ex}&gt;50$</td>
<td>See Fig</td>
<td>Excavate and replace 0.6m clay as Fig</td>
</tr>
<tr>
<td></td>
<td>Earthworks cover min 1m</td>
<td>Sealed shoulders</td>
</tr>
<tr>
<td></td>
<td>Side slopes 1:6 min</td>
<td>Shoulder width min 2m</td>
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<td></td>
<td>Alternative:</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td>None</td>
<td>Alternative:</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sealed shoulders</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Shoulder width min 2m</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Earthworks cover min 1m</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Side slopes min 1:6</td>
</tr>
</tbody>
</table>

* Where the earthworks cover is > 2m the side slopes can be made maximum 1:4
Processing and compaction of expansive soils does not reduce their swell properties and their strength is not significantly increased. Attempts to adjust the moisture content, e.g., to achieve an optimum are time-consuming, impractical, and unnecessary. Nominal rolling of the roadbed is desirable to obtain a working surface for construction of pavement layers. Fill materials used for replacement of expansive soil should meet the specifications for fill. Plastic soils with minimum PI of 15 should be used when available at economic haulage distances.

**Figure 11.2: Alternative methods of construction on Expansive Soil**

<table>
<thead>
<tr>
<th>Embankments 2m or higher</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pavement layers</td>
</tr>
<tr>
<td>Fill and improved subgrade 1:2</td>
</tr>
<tr>
<td>Freely draining material 1:6 or flatter</td>
</tr>
<tr>
<td>Expansive soil excavated 0.6m deep and replaced with fill</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Embankments less than 2m in height</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pavement layers</td>
</tr>
<tr>
<td>Fill and improved subgrade 1:2</td>
</tr>
<tr>
<td>Freely draining material 1:6 or flatter</td>
</tr>
<tr>
<td>Expansive soil excavated 0.6m deep and replaced with fill</td>
</tr>
</tbody>
</table>

11.3 SALINE SOILS

The presence of soluble salts, i.e., NaCl, Na₂CO₃, NaHCO₃, (but not gypsum, Na₂SO₄, which is only slightly soluble) in pavement or earthwork materials, or more critically in the subgrade and/or groundwater can cause damage to prime coats and thin surfacings. This is a significant risk in arid climates because of the migration of these salts to the surface as a result of evaporation. Coastal areas are most at risk from this mode of damage.
The content of soluble salt can be rapidly but indirectly determined by laboratory or field determination of electrical conductivity. It is prudent to determine the precise configuration of soluble salts by chemical analysis on a few samples and relate this to the conductivity. If the tests are carried out on potential pavement materials, rather than subgrade, construction water should be added at 1.5 times the required amount to obtain the OMC, to allow for evaporation, before the sample is tested.

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

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

11.4 ORGANIC SOILS

These commonly occur in swamp areas and require special investigations to evaluate ground stability and potential for excessive settlement. Typically, remediation consists of surcharging the pavement structure for a specified time before removing the surcharge and constructing the pavement. Other remediation measures comprise removal and replacement of the organic soil or, in extreme cases, construction of the road ‘floating’ on the swamp material.

A high content of organic matter (>2%) is undesirable in pavement materials, particularly in stabilized layers because it causes increased demand for stabilizer to achieve the required strength.
12 DESIGN OF GRAVEL AND LOW STANDARD ROADS

12.1 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 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 Somaliland 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.

12.2 DESIGN PRINCIPLES

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

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).

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 12-1.

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.
12.3 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 (D₁).
2. Determine the extra thickness needed to compensate for the gravel loss under traffic during the period between regravelling operations (D₂).
3. Determine the total gravel thickness required by adding the above two thicknesses (D₁ + D₂).

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.

GRAVEL LOSS

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

\[ GL = \frac{f T^2}{(T^2 + 50)} (4.2 + 0.092 T + 3.50 R^2 + 1.88V) \]

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 quartzitic gravels
  = 0.7 to 0.96 for volcanic gravels (weathered lava or tuff)
  = 1.5 for coral gravels
  = 1.38 for sandstone gravels
**TOTAL THICKNESS REQUIRED**

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

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

Where $D_1$ is the minimum thickness from Figure 12.3

$N$ 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 $D_1$.

12.4 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.

12.5 CROSSFALL AND DRAINAGE

The crossfall of carriageway and shoulders for gravel roads shall be “4%” as indicated in RDA’s *Geometric Design Manual - 2014*. 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.

12.6 MATERIAL REQUIREMENTS

**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.

**MARGINAL MATERIALS**

Figure 12-1 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).
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.

**SUBGRADE CBR**

All subgrade materials shall be brought to 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 12.7.

**TREATMENT OF EXPANSIVE FORMATIONS**

The following treatment operations should be applied on Expansive Formations for higher class roads of AADT\textsubscript{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\textsubscript{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.

\textit{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 12-1.
### Table 12-1

<table>
<thead>
<tr>
<th>Material Properties</th>
<th>G20 (Upper Layer)</th>
<th>G7 (Lower Layer)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CBR Dry Climatic Zones (See Note)</td>
<td>Minimum 20 after 4 days soaking</td>
<td>Minimum 7 after 4 days soaking</td>
</tr>
<tr>
<td>CBR Wet Climatic Zones (See Note)</td>
<td>Minimum 20 at OMC</td>
<td>Minimum 7 at OMC</td>
</tr>
<tr>
<td></td>
<td>Minimum 7 after 4 days soaking</td>
<td>Minimum 3 after 4 days soaking</td>
</tr>
<tr>
<td>PI [%]</td>
<td>Maximum 25</td>
<td>Maximum 30</td>
</tr>
<tr>
<td>Compacted Density</td>
<td>95% of AASHTO T180</td>
<td>95% of AASHTO T180</td>
</tr>
<tr>
<td>Maximum particle size</td>
<td>2/3 of layer thickness</td>
<td>2/3 of layer thickness</td>
</tr>
<tr>
<td>Compacted layer thickness</td>
<td>Maximum 200 mm</td>
<td>Maximum 250 mm</td>
</tr>
</tbody>
</table>

**Note:** Climatic Zones for Somaliland are described in Section 12.10.

12.7 **GRAVEL WEARING COURSE**

**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 12-1 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<sub>design</sub> 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<sub>design</sub> less than 50. Other alternatives are also specified in this chapter.
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 12-2.

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>
<thead>
<tr>
<th>Test Sieve Size (mm)</th>
<th>Percent (%) by mass of total aggregate passing test sieve</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Type 1</td>
</tr>
<tr>
<td>50</td>
<td>-</td>
</tr>
<tr>
<td>37.5</td>
<td>100</td>
</tr>
<tr>
<td>28</td>
<td>-</td>
</tr>
<tr>
<td>20</td>
<td>80-100</td>
</tr>
<tr>
<td>14</td>
<td>-</td>
</tr>
<tr>
<td>10</td>
<td>55-100</td>
</tr>
<tr>
<td>5</td>
<td>40-60</td>
</tr>
<tr>
<td>2.36</td>
<td>30-50</td>
</tr>
<tr>
<td>2</td>
<td>-</td>
</tr>
<tr>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td>0.425</td>
<td>15-30</td>
</tr>
<tr>
<td>0.075</td>
<td>5-15</td>
</tr>
</tbody>
</table>

**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 RDA Geometric Design Manual -2014, 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.

**Minor Gravel Roads (AADTdesign < 20)**

Minor gravel roads are roads which have a design AADT (AADTdesign) 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 RDA Geometric Design Manual-2014). 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.

12.8 DETERMINATION OF $C_B R_{\text{DESIGN}}$

**GENERAL**

The $C_B R_{\text{design}}$ 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 $C_B R_{\text{design}}$ is shown in the flow chart in Figure 12-2.

![Flow Chart for Design](image)

**Figure 12-2: Flow Chart for Design**

**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.
STATISTICAL ANALYSIS

The flow chart in Figure 2 shows the procedure to determine CBR\textsubscript{design}.

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

The CBR\textsubscript{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 \( d = 0.1 \times (n-1) \), where \( n \) = number of tests;
3. \( d \) is measured along the "x axis" and the CBR\textsubscript{design} is determined from the "y axis".

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).

12.9 IMPROVED SUBGRADE AND PAVEMENT DESIGN

MAJOR GRAVEL ROADS

Pavement and improved subgrade for major gravel roads shall be constructed in accordance with Figure 12-3. This includes all design categories DS5, DS6, DS7 and DS8 as defined in RDA Geometric Design Manual -2014.

MINOR GRAVEL ROADS

Pavement and improved subgrade for minor gravel roads shall also be constructed in accordance with Figure 12-3. This includes design categories DS9 and DS10 as defined in RDA Geometric Design Manual -2014. The desired properties of the gravel wearing course material, GW, are given in Section 12.7. 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.
12.10 CLIMATIC ZONES

For the purposes of gravel wearing course design, Somaliland is divided into two climatic zones. All places with elevations over 1,000 meters (average rainfall 40mm/month) are considered to be wet zones and all places with elevations 1,000 meters or less (average rainfall 15mm/month) are considered to be dry zones. However, engineering judgement should be made for individual projects as to which category the design falls.

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.
13 CONCRETE ROADS

13.1 INTRODUCTION

The purpose of this chapter is to give specific guidance and recommendations to the engineers responsible for the design of rigid pavements in Somaliland.

This chapter contains:
- A description of rigid pavements: their characteristics, their components and their function, the different types of slabs and joints, including drawing details.
- A description of the factors influencing the pavement type selection and the design process.
- A design procedure for the different types of pavement, slab reinforcement, joint details and joint layout.

The design method is a directly utilizable one, based mainly on empirical results and full scale experiments. Although an analytical, comprehensive approach to the design is possible, based on the stresses and strains induced in the pavement by an applied wheel loading, it is very complicated, rarely used, leads to minor changes and as such is not covered in these pages.

13.2 RIGID PAVEMENTS

13.2.1 GENERAL CHARACTERISTICS

Rigid pavements (concrete pavements), as the name implies, are rigid and considerably stronger in compression than in tension. One of the main characteristics of rigid pavements is that a relatively thin pavement slab distributes the load over a wider area due to its high rigidity. Localized low strength roadbed material can be overcome due to this wider distribution area. In concrete pavements, the strength of the pavement is contributed mainly by the concrete slab, unlike flexible pavements where successive layers of the pavement contribute cumulatively.

Concrete pavements are subject to thermal stresses due to variation in daily and annual temperatures. In Somaliland, though the annual range of temperature is small, the daily range of temperature is high, varying from 20°C to 40°C (Ref. 1), and hence thermal stresses deserve attention for the design of concrete pavements.

Skidding resistance of concrete pavement surfaces is also an important functional characteristic. The rugose surface required for an adequate resistance to skidding in wet conditions can be provided by dragging stiff brooms transversely across the newly-laid concrete or by cutting shallow randomly spaced grooves in the surface of the hardened concrete slab.

In the absence of deleterious materials (either in the aggregate or entering the concrete in solution from an external source), unlike with flexible pavements, concrete does not
suffer deterioration from weathering. Neither its strength nor its stiffness is materially affected by temperature changes. At present, concrete pavements have not been extensively used in most tropical countries and in Somaliland in particular, mainly due to a lack of tradition and experience in design and construction. One characteristic of concrete pavements is that either they prove to be extremely durable, lasting for many years with little attention, or they give trouble from the start, sometimes because of faults in design, but more often because of mistakes in construction.

13.2.2 TYPES OF RIGID PAVEMENTS

Depending on the level of reinforcement, the rigid pavements can be categorized into three basic types:

- Jointed unreinforced concrete pavements (JUCP)
- Jointed reinforced concrete pavements (JRCP)
- Continuously reinforced concrete pavements (CRCP)

In jointed unreinforced concrete pavements (JUCP), the pavement consists in a succession of cast in place unreinforced concrete slabs separated by joints to prevent expansion from developing stresses and to control cracks. The slabs are linked together by tie bars or dowels to transmit the vertical stresses.

In jointed reinforced concrete pavements (JRCP) the pavement consists in a succession of cast in place reinforced concrete slabs separated by joints to prevent expansion from developing stresses and to control cracks. The slabs are linked together by tie bars or dowels to transmit the vertical stresses. JRCP are used where a probability exists for transverse cracking during pavement life due to such factors as soil movement and/or temperature/moisture change stresses. The longitudinal reinforcement is the main reinforcement. A transverse reinforcement though not absolutely necessary in most cases is usually added to facilitate the placing of longitudinal bars.

Continuously reinforced concrete pavements (CRCP) are used for rather trafficked roads where a good level of comfort is expected. The principal reinforcement, in the form of prefabricated mesh or reinforcing bars installed at mid-depth of the slab, is again the longitudinal steel which is essentially continuous throughout the length of the pavement. This longitudinal reinforcement is used to control cracks which form in the pavement due to volume change in the concrete.

13.3 PAVEMENT STRUCTURE, FUNCTION OF PAVEMENT LAYERS AND COMPONENTS

Rigid pavements generally consist of, as shown in Figure 1, a subbase, and a concrete slab constructed above the subgrade constituted of the roadbed or embankment and a capping layer (if required).

The capping layer consists of selected fill and is provided in cases of low strength roadbed material. It protects the underlying subgrade from construction traffic loading and provides a stronger platform for the subbase layer, which is placed on top of the capping layer.
The subbase of a rigid pavement structure consists of one or more compacted layers of material placed between the subgrade and the rigid concrete slab. In some cases the material can be cement stabilized to increase its quality. If the roadbed soils are of an acceptable quality and if the design traffic is low (less than one million equivalent standard axles (ESAs)), a subbase layer may not be necessary between the prepared roadbed and the concrete slab.

A subbase is provided under a concrete pavement for the following reasons:

- to provide a stable “working platform” for the construction equipment;
- to provide a uniform concrete slab support;
- to help in the control of excessive volume changes in roadbed soils susceptible to such phenomena; and
- to prevent “pumping” at joints and slab edges.

The concrete slab consists of Portland cement concrete, reinforcing steel (When required), load transfer devices and joint sealing materials.

Transverse reinforcement is provided to ensure that the longitudinal bars remain in the correct position during the construction of the slab. It also helps to control any longitudinal cracking that may develop.
The details regarding the design of the pavement slab thickness and the amount of reinforcement required are discussed in Chapter 7.

### 13.4 JOINTS

Joints are placed in concrete pavements, whether reinforced or not to permit expansion and contraction of the pavement, thereby relieving stresses due to environmental changes (Temperature and moisture), friction, and to facilitate construction. There are four general types of joints: contraction, expansion, warping and construction.

In most cases joints combine several of these functions:

- Contraction joints are provided to relieve the tensile stresses due to temperature, moisture and friction, therefore controlling cracking. If contraction joints were not installed, random cracking would occur on the surface of the pavement.

- The primary function of an expansion joint is to provide space for the expansion of the pavement, thereby preventing the development of compressive stresses, which cause the pavement to buckle. Expansion joints are also contraction joints.

- Warping joints allow a slight relative rotation of the slab portions delimited by joints and reduce the strains due to warping. Warping joints are also contraction joints. Longitudinal joints are always warping joints but they can be found also as transverse joints in some cases (See further).

- Construction joints are required to facilitate construction, especially when concreting is stopped. For JUCP and JRCP, they shall be coupled with other joints and additional reinforcement shall be placed when dealing with transverse construction joints for CRCP.

Load transfer between slabs is provided by dowels. All materials required for joints and sealing shall be as per the Standard Specifications.

Joints can be broadly classified into two categories according to their direction: (1) Transverse Joints and (2) Longitudinal Joints.

Typical details of the joints are presented in Appendix B (adapted from Ref. 8).

**Transverse Joints**

Based on their function, the transverse joints can be divided into three types: (i) contraction joints, (ii) expansion joints and (iii) warping joints.

**Contraction Joints**

Contraction joints are the main type of transverse joint. They provide weakened sections between slabs to induce tension cracking at preferred locations in the concrete after it has been placed. They also contribute to limiting the strain due to warping as a result of temperature and moisture changes. Contraction joints shall consist of:

- a sawn joint groove
- dowel bars
- a sealing groove

The groove and sealant shall be as specified. The dowel bars shall be 20 mm in diameter at 300 mm spacing, 400 mm long for slabs up to 239 mm thick, and 25 mm in diameter for slabs 240 mm thick or more.

**Expansion Joints**

Transverse expansion joints are required in JUCP and JRCP at regular intervals. The distance required between joints depends on the expected expansion and on the friction between the different layers of the pavement.

In expansion joints, complete separation between the two adjacent concrete slabs is required, and a compressible joint is used to fill the void.

Expansion joints shall consist of:

- a joint filler board
- dowel bars
- a sealing groove

The joint filler board and sealing groove shall be as per the Specifications. The dowel bars shall be 25 mm in diameter at 300 mm spacing, 600 mm long for slabs up to 239 mm thick, and 32 mm in diameter for slabs 240 mm thick or more.

**Warping Joints**

Transverse warping joints are used for special cases, such as extra joints at manhole positions, or when unreinforced slabs are alongside reinforced slabs, or in long and narrow or tapered (odd-shaped) JUCP slabs between normal joint positions, to reduce the length/width ratio of the slabs to 2 or less, and in other similar situations.

Warping joints shall consist of:

- a sawn groove
- tie bars
- a sealing groove

The sealant shall be as per the Specifications. The tie bars shall be 12 mm in diameter at 300 mm spacing, and 1000 mm long.

**Longitudinal Joints**

Longitudinal joints are warping joints, required at such a spacing as will reduce the combination of thermal warping stresses and loading stresses to a minimum, and reduce the risk of longitudinal random cracking, and often serve at the same time as construction joints. These joints allow a slight rotation, but differential lateral displacements between adjacent slabs are prevented by tie bars provided at mid-depth of the slab.
Longitudinal joints shall consist of:

- bottom crack inducer
- a sawn groove
- tie bars
- a sealing groove

The sealant shall be as per the Specifications. The tie bars for all longitudinal joints, except where transverse reinforcement is permitted in lieu, shall be 12 mm in diameter at 600 mm spacing, and 1000 mm long.

13.5 SELECTION OF PAVEMENT TYPE

The highway engineer or administrator does not have at his disposal an absolute or indisputable method for determining the type of pavement which should be selected for a given set of conditions.

First a judgement must be made on many varying factors such as traffic, soil, weather, materials, construction, maintenance and environment. In some cases overriding factors can dictate pavement type. For instance, for heavily traveled facilities in congested locations, the need to minimize the disruptions and hazard to traffic may dictate the selection of CRCP.

When there is no overriding factor, which may often be the case, it is standard practice to design typical sections of the road using each of the available options and then to compare them on an economical point of view.

Unavoidably, there will be instances where financial circumstances are such to make first cost the dominate factor in selection, even though higher maintenance or repair costs may be involved at a later date.

Where circumstances permit, a more realistic economical evaluation has to take into account all expected costs including the initial cost of construction, the cost of subsequent stages or corrective works, anticipated life, maintenance cost and salvage value. Costs to road users during periods of reconstruction or maintenance operations are also appropriate for consideration.

Although pavement structures are based on an initial design period, few are abandoned at the end of this period and continue to serve as part of the future pavement structure. For this reason, the analysis period should be of sufficient duration to include a representative reconstruction of all pavement types.

If the analysis of the above given factors does not show a far higher interest of one option rather than another, a second set of factors can be considered such as the performance of similar pavements in the area or the skills of contractors.

Basically, the use of the different types of rigid pavement is as follows:

- JUCP is suitable for all levels of traffic, whenever the risk of subgrade movement is low and an uncontrolled cracking not very prejudicial.
- JRCP is suitable for all levels of traffic and is used when the risk of settlements of the subgrade can not be neglected.
- CRCP shall basically be considered only for rather high design traffic (>30 msa).
They can also be included for less heavily trafficked schemes where the advantage of lower maintenance throughout the design life may be worthwhile.

13.6 STRESS DEVELOPMENT AND DESIGN CRITERIA

13.6.1 STRESS DEVELOPMENT

The concrete slabs in concrete pavement are subjected to two main types of stresses, the stresses developed because of changes of the environment (moisture and temperature) and closely depending on the intrinsic properties of the concrete and the stresses coming from the traffic loads.

These kinds of stresses can not be prevented from developing, but the design of concrete pavement shall take them into account in order to keep them in acceptable ranges of values. Analytical methods have been developed aiming at computing accurately the stresses developed in concrete pavements for a theoretical design. But they are rather complicated and necessitate strong hypothesis on the quality and the evolution of the different layers composing the road bed. That’s why the approach of this manual is limited to a qualitative description of the phenomena which justify the pragmatic design procedure exposed in section 7.

**Horizontal Tensile stresses**

While setting and later depending on external humidity, the moisture rate of the concrete changes, thus inducing tensile stresses in the material.

If the temperature drops after setting, thermal tensile stresses are also induced, depending on the thermal coefficient of the concrete.
The acceleration or slowing down of vehicles also induces horizontal tensile stresses in the pavement, because of the mechanical reaction.

Since the movements of the lower face of the pavement are limited by the friction with the subbase, cracks appear as soon as the sum of these tensile stresses exceed the concrete tensile strength. When uncontrolled cracks become too wide, they enable water infiltration or slabs and subgrade vertical movements thus accelerating the degradation of the pavement.

Depending on the type of slab this issue is treated differently:

In JUCP and JRCP:
The development of cracks is controlled by placing joints at regular intervals and a separation membrane between the slab and the subbase. The limited lengths of slabs and the increased possibility of horizontal movements limit the tensile stresses and thus prevent the slabs from cracking between joints.

In CRCP:
The continuous reinforcement causes the cracking to occur at regular and little spaced locations thus limiting the width of cracks to acceptable values.

**Horizontal compressive stresses**

If the temperature rises after setting, thermal compressive stresses are also induced, depending on the thermal coefficient of the concrete.

The acceleration or slowing down of vehicles also induces horizontal compressive stresses in the pavement, because of the mechanical reaction.

If the sum of these compressive strengths become to high, this may cause the pavement to buckle.

According to the type of slab, this issue is treated as follows:

In JUCP and JRCP:
The placing of expansion joints and the increased possibility of movement through the use of the separation membrane permit the expansion of the concrete and the dissipation of compressive stresses.

In CRCP:
The continuous reinforcement increases the resistance of slabs, eliminates week sections and prevents any buckling.

**Vertical stresses**

With a minimum concrete thickness of 150 mm, vertical stresses developed by the traffic loads are prejudicial for rigid pavement only because of their repetitive character causing the progressive degradation of the underlying layers. Indeed, the punctual loss of support and the movements of the subgrade are the main reasons for rigid pavement not to bear vertical stresses.
13.6.2 DESIGN CRITERIA

As above explained, the factors which shall intervene in the design of rigid pavements are as follows:

- the roadbed quality
- the quality of the steel and concrete composing the slabs
- the traffic
- the environment. (Moisture and temperature)

Another criterion which usually intervenes in the design of road pavement, the design period is not considered here because rigid pavements are deemed suitable for long design periods, which means 20 years.

For the simplified experience-based design procedure exposed in this manual, the assumption is made that the materials used for construction meets the standard requirements as defined in section 3. Which means mainly a yield strength of 500 MPa for the steel reinforcement bars and a 28 day characteristic compressive strength of 40 MPa for the concrete.

As stated in the previous section, the accurate computation of the stresses in the concrete is not in the scope of this manual and thus, the proposed thickness design procedure and joint layout for rigid pavement is suitable for the usual natural ranges of temperature and moisture rates.

Consequently, the two parameters to be accounted for in the design procedure are the traffic data and the bearing capacity of the roadbed.
**Equivalency Factors – Cumulative Equivalent Standard Axles (ESAs)**

Refer to Chapter 3, for details for computing equivalency factors and the cumulative ESAs.

**Table 1**

Equivalency Factors for Different Axle Loads (Rigid Pavements)

<table>
<thead>
<tr>
<th>Wheel load (single and dual) $(10^3 \text{ kg})$</th>
<th>Axle load $(10^3 \text{ kg})$</th>
<th>Equivalency factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>3.0</td>
<td>0.02</td>
</tr>
<tr>
<td>2.0</td>
<td>4.0</td>
<td>0.05</td>
</tr>
<tr>
<td>2.5</td>
<td>5.0</td>
<td>0.13</td>
</tr>
<tr>
<td>3.0</td>
<td>6.0</td>
<td>0.28</td>
</tr>
<tr>
<td>3.5</td>
<td>7.0</td>
<td>0.53</td>
</tr>
<tr>
<td>4.0</td>
<td>8.0</td>
<td>0.93</td>
</tr>
<tr>
<td>4.5</td>
<td>9.0</td>
<td>1.53</td>
</tr>
<tr>
<td>5.0</td>
<td>10.0</td>
<td>2.40</td>
</tr>
<tr>
<td>5.5</td>
<td>11.0</td>
<td>3.63</td>
</tr>
<tr>
<td>6.0</td>
<td>12.0</td>
<td>5.25</td>
</tr>
<tr>
<td>6.5</td>
<td>13.0</td>
<td>7.33</td>
</tr>
<tr>
<td>7.0</td>
<td>14.0</td>
<td>9.92</td>
</tr>
<tr>
<td>7.5</td>
<td>15.0</td>
<td>13.1</td>
</tr>
<tr>
<td>8.0</td>
<td>16.0</td>
<td>17.0</td>
</tr>
<tr>
<td>8.5</td>
<td>17.0</td>
<td>21.6</td>
</tr>
<tr>
<td>9.0</td>
<td>18.0</td>
<td>27.1</td>
</tr>
<tr>
<td>9.5</td>
<td>19.0</td>
<td>33.7</td>
</tr>
<tr>
<td>10.0</td>
<td>20.0</td>
<td>41.4</td>
</tr>
</tbody>
</table>

The equivalency factors for rigid pavements for different axle loads are given in Table 1. These factors are marginally higher, when compared with the corresponding values from Table 2.3, for loads up to an axle load of $9 \times 10^3$ kg. However, for heavier loads, the equivalency factors for rigid pavements are lower and the difference increases sharply with higher loads. For rigid pavements, by adopting the equivalency factors from Table 2.1, the resultant cumulative ESAs may be much higher, particularly for traffic with a high percentage of heavy trucks. Hence, the design engineer should evaluate the sensitivity of the results prior to deciding which factors should be used in calculating cumulative ESAs.

**Subgrade Assessment**
The strength of the subgrade is assessed in terms of the California Bearing Ratio (CBR). Refer to Chapter 3.2 for subgrade assessment.

### 13.7 DESIGN OF RIGID PAVEMENTS

A general methodology of rigid pavement design is presented in Figure 2.

#### 13.7.1 DESIGN TRAFFIC LOADING

Refer to Chapter 3 and to subsection 6-2 for the computation of the design Equivalency Axle Load of the road.

#### 13.7.2 THICKNESS DESIGN

**Capping and Subbase**

The capping layer is required only if CBR of the subgrade is 15% or less. The required thickness of a capping layer for a CBR value less than 15% can be obtained from Figure 3.

The subbase layer is required when the subgrade material doesn’t comply with the requirement for a subbase (CBR is less than 30%) or to facilitate the obtaining of the surface levels with the tolerances required. Generally, the thickness of the subbase provided will be a constant 15 cm and can be cement stabilized.

For subgrade CBR values less than 2%, the roadbed material needs to be treated either by replacement or in-situ stabilization. These methods of soil improvement are described in section 7 of the flexible pavement design manual.

A separation membrane (such as a polythene sheet) is required between subbase and concrete slab, mainly in order to reduce the friction between the slab and the subbase in JUCP and JRCP pavements, and thus inhibits the formation of mid-bay cracks. It also reduces the loss of water from the fresh concrete. For CRCP pavements, a bituminous spray should be used on the subbase, instead of polythene, because a degree of restraint is required.

**Concrete Slab Thickness and Reinforcement**

Based on the design traffic volume determined as per Chapter 2 and project-specific characteristics, the thickness of pavement is determined.

The following represents procedures for determining the thickness and reinforcement for each of the pavement types.

**Jointed Unreinforced Concrete Pavement (JUCP)**

For a given traffic volume in terms of ESAs, the thickness of JUCP concrete slab can be determined using Figure 4.
Figure 4 assumes the presence of an effective lateral support to the edge of the most heavily-trafficked lane (i.e., the right lane), such as a shoulder with a pavement structure able to carry occasional loads. In the absence of such a shoulder adjacent to the most heavily trafficked lane, an additional slab thickness is required, and this additional thickness can be determined using Figure 6.

JUCP pavements have no reinforcements. However, the longitudinal and transverse joints are provided with reinforcements. The joint details are discussed in Section 5 Jointed Reinforced Concrete Pavement (JRPC)

For a given traffic volume in terms of ESAs, the thickness of a JRCP concrete slab can be determined using Figure 4. The figure can also be used to determine the longitudinal reinforcement in terms of mm²/m for a design thickness of concrete slab. Thus, several alternate combinations of thickness of concrete slab and amount of reinforcement can be compared.

In the absence of an effective lateral support provided by the shoulder adjacent to the most heavily trafficked lane, an additional slab thickness is required and can be determined using Figure 6.

In addition to the longitudinal reinforcement, JRCP pavements shall be provided with transverse reinforcement, if required, depending on site conditions. In that case, reinforcement shall be provided at 600 mm spacing and consist of 12 mm diameter steel bars.

Continuously Reinforced Concrete Pavement (CRCP)

CRCP pavement can withstand severe stresses induced by differential movements. For a given traffic volume, in terms of ESAs, the thickness of CRCP concrete slab can be obtained from Figure 5.

Longitudinal reinforcement in CRCP pavements shall be 0.6% of the concrete slab cross-sectional area, consisting of 16 mm diameter deformed steel bars. If required, transverse reinforcement shall be provided at 600 mm spacings, consisting of 12 mm diameter deformed steel bars, to control the width of any longitudinal cracks which may form. Transverse reinforcement is normally required only for ease of construction. It may be omitted except where there is a risk of differential settlements.

Similarly to JUCP and JRCP pavements, in the absence of effective shoulder support adjacent to the most heavily trafficked lane, the additional slab thickness required can be determined using Figure 6.

As is evident from Figure 4 and 5, the minimum thickness of concrete pavement for JUCP and JRCP pavement is 150 mm and that for CRCP pavement is 200 mm. Hence, the designer should carefully assess the necessity and requirements for such pavements, depending on the design traffic volume, and shall include flexible pavement as an alternate.

A design example of rigid pavement design is presented in Appendix C.

13.7.3 DESIGN FOR MOVEMENT
Joints shall be designed according to the general considerations of sub section 2.2 and using Drawings B2 to B8.

The general layout of joints shall account for construction consideration and the following limitations concerning joint spacing and slabs dimensions:

**Transverse Joint Spacing**

Maximum transverse joint spacing for JUCP pavements is 4 m for slab thickness up to 230 mm and is 5 m for slab thickness over 230 mm.

Expansion joints should replace every third contraction joint, ie. at a spacing of 12m or 15m.

For JRCP, contraction joints are generally at a standard distance of 25m, unless there is 500mm$^2$/m of reinforcement. Then refer to Figure 4. For expansion joints, replace every third contraction joint with an expansion joint. For example, a pavement with contraction joint spacing of 25m has an expansion joint spacing of 75m.

**Longitudinal joint spacing**

The longitudinal joint spacing shall not be greater than 4.2 m for pavement slabs without transverse reinforcement and 6.0 m for pavement slabs with transverse reinforcement. When required, longitudinal joints shall be placed at the edge of traffic lanes.

**Slabs dimensions**

Warping joints shall be added to the general layout in special cases, as described in section 2.2. In general, the length/width ratio of the slabs shall be of 2 or less.

**Layout at junctions**

For intersection of joints at junctions and crossings, refer to the drawing below:
Intersection of transverse joints

Longitudinal joints  Transverse joints

Joints layout at junctions and crossings
Figure 2  Design Methodology – Flow Diagram
Figure 3  Capping Layer and Subbase Thickness design  
(Adapted from Ref.5)
Figure 4    Design Thickness and joint spacing for JUCP and JRCP
(Adapted from Ref. 5)
Figure 5  Design Thickness for CRCP Pavements  
(Adapted from Ref. 5)

Figure 6  Additional Concrete Slab Thickness for Rigid Pavements 
Without Lateral Support (from Ref. 5)
14 PAVEMENT REHABILITATION AND ASPHALT OVERLAY DESIGN

14.1 INTRODUCTION

14.1.1 PURPOSE AND SCOPE OF THE CHAPTER

PURPOSE

The Roads Development Agency (RDA) has initiated a comprehensive program to rehabilitate and upgrade the highway network in Somaliland.

As the network of paved roads in Somaliland evolves, it is likely that a gradual shift in emphasis will occur, from new design and construction to maintenance and rehabilitation of the existing paved network.

The purpose of this chapter is to provide, as a complement, a guide for the design of the most common rehabilitation solutions applicable to paved roads, and for the selection of an appropriate rehabilitation alternative for a particular project.

SCOPE

This chapter provides design procedures for the rehabilitation of both flexible and rigid pavements. The procedures utilize data which are collected during desk studies as well as by means of field work.

The data and their analysis are primarily directed towards a rehabilitation design on an individual project basis, but are also meant to be used, on a more general basis, as inputs to RDA’s Pavement Management System (PMS).

Several rehabilitation alternatives, including asphalt overlays, are usually available to the design engineer from a technical viewpoint; the manual provides guidance regarding the choices, practical and economical, between the various alternatives.

14.1.2 THE PAVEMENT MANAGEMENT CONTEXT

RDA’S PAVEMENT MANAGEMENT SYSTEM

Pavement management, in its broadest sense, encompasses all the activities involved in the planning, design, construction, maintenance, evaluation and rehabilitation of the pavement portion of a public works program. A pavement management system (PMS) is a set of tools or methods that assist decision makers in finding optimum strategies for providing, evaluating, and maintaining pavements in a serviceable condition over a given period of time. The function of a PMS is to improve the efficiency of decision-making, expand its scope, provide feedback on the consequences of decisions, facilitate the coordination of activities within the agency, and ensure the consistency of decisions made at different management levels within the agency.

In this sense, pavement “design”, as covered by RDA’s above-mentioned manual as covered in this manual, are essential parts of the overall pavement management process, a conceptual illustration of which is given in Figure 1.2 (after Ref. 3 and 4).
As can be seen on Figure 1.2, it is convenient and customary to describe a PMS in terms of two generalized levels: (1) the network management level, and (2) the project level.

**Figure 1.2  Activities of a Pavement Management System (after Ref. 3 and 4)**

At the network level, the pavement management system gives information relevant to the development of programs involving the road infrastructure considered as a group of itineraries, identified by their functional class and their main technical characteristics and level of traffic. Basically, the pavement management system deals primarily with network level where key administrative decisions that affect programs for road networks are made. It allows one to evaluate and compare series of maintenance activities and maintenance and rehabilitation alternatives, and to test them with regard to budget constraints.

The project level is the appropriate level for evaluating the detailed pavement conditions of a section of an itinerary, and for designing technical solutions for routine, periodic
maintenance, or rehabilitation, according to road management priorities, selected cost-efficient policy alternatives, and budget constraints. Here again, by comparing the benefits and costs associated with several alternative activities, an optimum strategy is identified that will provide the desired benefits at the least cost over a selected analysis period.

**MAINTENANCE AND REHABILITATION**

A list of terms and abbreviations used in this manual is presented in Appendix A. However, two terms, “maintenance” and “rehabilitation,” deserve particular attention in order to fully understand, and better benefit from, this manual. As already indicated, the PMS aims at allocating resources between maintenance and rehabilitation, and an outline of the method used by RDA’s PMS for this purpose is given in Chapter 4. It is therefore appropriate to define what is covered by these two groups of activities.

**MAINTENANCE**

The following excerpt from the introduction to the Asphalt Institute Manual Series No. 16, *Asphalt in Pavement Maintenance* (Ref. 6) is considered appropriate to define maintenance, as well as to provide some insight into two general categories of maintenance:

“Pavement maintenance is not easy to define. Maintenance departments generally agree what it is, but there are some minor differences. Some call pavement improvement “maintenance”. Others include only the work that keeps the pavement in its as-constructed condition.

Taking all these into consideration, the definition that seems to fit best is:

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

Distinctions are usually made between forms of maintenance, based on their required frequency. The International Road Maintenance Handbook (Ref. 7) uses the grouping of “routine” and “periodic” maintenance, while other sources (e.g. TRRL, Ref. 8) use “routine”, “recurrent”, “periodic” and “urgent”. The following excerpt from Reference 8 illustrates these categories:

“… There are four categories:

- **routine** maintenance, required continually on every road, whatever its engineering characteristics or traffic volume
- **recurrent** maintenance, required at intervals during the year with a frequency that depends on the volume of traffic using the road
- **periodic** maintenance, required only at intervals of several years
- **urgent** maintenance, needed to deal with emergencies and problems calling for immediate action when a road is blocked.

Examples of activities within these categories are as follows:

**routine:**
grass cutting; drain clearing; recutting ditches; culvert maintenance; road signs maintenance

**recurrent on paved roads:**
repairing pot-holes; patching; repairing edges; sealing cracks

**periodic on paved roads:**
resealing (surface dressing, slurry sealing, fog spray, etc.); regravelling shoulders; road surface marking

**urgent:**
removal of debris and other obstacles; placement of warning signs and diversion works.”

**REHABILITATION**

The following excerpt from the AASHTO Guide (Ref. 3) illustrates the essential difference between maintenance and rehabilitation:

“Major rehabilitation activities differ markedly from periodic maintenance activities (sometimes called normal, routine and/or preventive maintenance) in that the primary function of the latter activity is to preserve the existing pavement so that it may achieve its applied loading, while rehabilitation is undertaken to significantly increase the functional life.”

In this manual, the following definition of rehabilitation, which also introduces the major rehabilitation categories, will be retained:

“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.”

Rehabilitation is the subject of this manual, and is given further scrutiny hereunder.

**REHABILITATION ALTERNATIVES**

In the context of pavement management at a project level, and as noted earlier, several rehabilitation alternatives are usually available after technical evaluation, and will need to be compared. It is therefore appropriate, before a more detailed description of these alternatives is given, to outline their main characteristics and the classification used in this manual.

For simplicity, rehabilitation is subdivided into two major categories:

1. Rehabilitation Methods with Overlays
2. Rehabilitation Methods other than Overlays

It is realized that various agencies define overlays in different ways, e.g. some sources would consider the addition of unbound courses as overlay. For clarity, and consistent
with the scope of this manual, the following definition of an overlay will be adopted for use in this manual:

“One or more courses of asphalt construction on 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.”

Bearing in mind the distinction between maintenance and rehabilitation, thin overlays will not be emphasized in the manual, nor will resurfacing with surface dressing or overlays of short (spot) length. It is noteworthy that asphalt concrete overlays of 5 cm are considered periodic maintenance in the context of the Pavement Management System (Ref. 5) on which RDA’s PMS is based.

It is also realized, and it should be recognized, that some of the methods which fall under “rehabilitation methods other than overlays” may be used/required as pre-overlay treatments in major rehabilitation work.

Rehabilitation methods other than overlays include, as broad primary categories:

- Reconstruction: in this category, little or no contribution is expected from the existing pavement materials and the materials needed for rehabilitation will be mostly new materials.
- Recycling: the rehabilitation takes advantage of the existing pavement materials, which are re-used in part or as a whole, in the construction of the rehabilitated pavement.

The above categories are general in nature, and some combinations of methods are possible and indeed used. For instance, the materials of an existing surface treatment and roadbase may be recycled as the roadbase or subbase of a reconstructed pavement, with new materials brought on site to construct the upper layers, e.g. roadbase and AC surface.

In addition to the above primary categories of non-overlay rehabilitation methods, other rehabilitation techniques (some of them, as noted, being also applicable prior to an overlay) can be applied to pavements to significantly extend their lives without the placement of an overlay, or may delay recycling or reconstruction for several years. These techniques, however, in order to truly qualify as rehabilitation techniques, must satisfy several criteria:

1) They must be applied only to pavements which are structurally adequate to support future traffic loadings over the design period without structural improvement from an overlay. Only structurally adequate pavements, or pavements restored to a structurally adequate state, are candidates for rehabilitation without overlay.
2) They must address the cause(s) of the pavement distress and be effective in both repairing existing deterioration and preventing its recurrence. For this, a combination of techniques may be required (one repair method and one preventive technique).

If each of the repair and preventive methods meet the pavement’s needs and satisfy the imposed constraints (such as available funding and minimum life extension), then they qualify as feasible rehabilitation alternatives.

If the alternative considered fails to satisfy the above criteria, it will be better classified under the term of maintenance.
Examples of major rehabilitation methods that may be employed as non-overlay techniques include:

1. Full-Depth Repair
2. Partial Depth Patching
3. Joint-Crack Sealing
4. Subsealing-Undersealing
5. Grinding and Milling
6. Subdrainage
7. Pressure Relief Joints
8. Load Transfer Restoration
9. Surface Treatments

These methods are defined and described further in Chapter 5 of the manual.

Finally, non-overlay rehabilitation methods also include, as required when these elements have become deficient, geometric improvements and/or drainage improvements or restoration.
14.2 DATA COLLECTION

In this chapter, general guidance is given relative to the gathering of data other than the data which needs to be collected in the field.

It is recommended that this collection and review of data be conducted prior to the field activities, as it can influence the latter with regard to their scope and organization.

The collection includes a search through the PMS Branch records, as detailed below. Conversely, the results of this collection, to the extent that they can update and/or complete the PMS Branch records, should be made available to the PMS Branch.

14.2.1 PMS BRANCH RECORDS

These records are among the first sources of data to be sought and collected. Normally, the data required for the PMS are regularly updated. They fall into three categories, as follows:

1) Data specifically collected for the PMS. This data deals with road inventory and road conditions. It includes the results of visual inspection, roughness and deflection surveys, and geotechnical investigations.

The visual inspection includes information about road geometry, road inventory features (length; width; profile; alignment; type of structures encountered; etc.), and about road damage (deformation; cracks; potholes; rutting; gullies; etc.). This information is extremely valuable in assessing the road condition; it is usually gathered along the entire length of the road.

Roughness is normally measured using a Bump Integrator and expressed through the International Roughness Index (IRI). Typical values of the IRI, depending upon the type of road and its condition, are as follows:

Table 2.1 Condition of the Road vs. IRI (from Ref. 5 and 9)

<table>
<thead>
<tr>
<th>IRI Ranges</th>
<th>Road Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lower than 6</td>
<td>very good</td>
</tr>
<tr>
<td>6 to 11</td>
<td>good</td>
</tr>
<tr>
<td>11 to 15</td>
<td>fair</td>
</tr>
<tr>
<td>15 to 19</td>
<td>poor</td>
</tr>
<tr>
<td>Larger than 19</td>
<td>very poor</td>
</tr>
</tbody>
</table>

While the road roughness is not directly involved in the structural design of the rehabilitation procedures presented in this manual, it may influence the choice and the method of effecting one alternative, e.g. by indicating a probable need for a leveling course prior to applying a structural overlay course.

Deflection surveys for the PMS are usually carried out using a Benkelman beam. Except for a variation in the magnitude of the load application, the procedure is similar to that described in this manual in Appendix B, and the user of this manual
should refer to these sections for details. A review of the PMS deflection data and a
comparison with the results of the deflection survey undertaken for the specific
project should give insight into the evolution of the load carrying capacity of the
pavement and/or increased confidence in the results of the specific survey.

Data specifically collected for the PMS also include detailed information about the
subgrade, pavement materials, and pavement structure. This information would have
been collected on sample sections by means of test pits with retrieval of samples for
laboratory analysis. The data from the test pits would normally include:

  a) Layers of the pavement structure: type, thickness, and maximum size of
aggregates of each layer.
  b) Subgrade: in-situ dry density; in situ moisture content; gradation; Atterberg
limits; classification according to AASHTO and/or USCS systems; and in situ
CBR obtained from correlation with DCP testing.

Here again, this data can be compared, and be a useful complement, to the project
specific geotechnical data collected in the field as detailed in Appendix B.

2) Data required for the PMS, but that has usually been collected separately in the field
and then forwarded to or obtained by the PMS Branch. This essentially includes
traffic and axle load data.

The data should normally include AADTs, classification of the traffic among the
various vehicle categories, and axle load data within each category. The data
available from the PMS Branch may already have been processed and be readily
available only in terms of traffic classes and cumulative equivalent axles loaded at 10
tonnes. It will then be necessary to collect the raw data in order to reprocess them
according to the procedure detailed further in this chapter (Section 2.3).

The usefulness of the traffic data thus collected will depend greatly on whether they
are sufficiently up-to-date. Nevertheless, they should be a valuable complement to
the evaluation of the traffic made for the specific project under consideration. In some
cases, they may be sufficiently recent, reliable, and complete to be used, at least in
part, for the project.

3) The third category of data normally available from the PMS Branch includes archive
data such as theoretical pavement structures; ages of the pavement structures; unit
prices of road works; vehicle operating costs and geo-climatic data.

General information about pavement structure and history should normally have been
made available to the PMS through RDA’s office and includes:

  - number and thickness of the pavement layers (surfacing, roadbase and
subbase), together with a description of the material of each layer (asphalt,
concrete, surface treatment, crushed basalt aggregates, etc.)
  - the date of construction or rehabilitation of the pavement structure
  - the type and date of major periodic maintenance activities

This data is very important to properly assess the road condition.
Unit rates of road works should also be available among the data in the PMS records. These costs will be useful in the economic comparison of the feasible rehabilitation alternatives, as detailed in Chapter 6 of this manual.

Vehicle operating costs (VOC) should not normally have an impact on the design of a specific project already selected for rehabilitation.

14.2.2 OTHER DATA

A review of existing documents, in addition to those available from the PMS Branch, should be conducted prior to field activities.

Such a review includes essentially a desk study similar to that undertaken for the feasibility study of any road project.

The geological environment of the project should be researched, together with the geotechnical characteristics corresponding to the geological formations along the alignment.

Sources of information regarding past investigations, published or not, are to be gathered and reviewed. They may include activities only loosely related to road construction concerns (e.g. agricultural, hydrology, mining).

Existing reports, maps, mineral resource surveys, boring logs, and any relevant data need to be collected and compiled. This is particularly valuable in identifying road construction materials.

Of particular importance, when available, are as-built plans of the road section under consideration or of adjacent projects, together with any memoirs or design reports (including pavement design reports and/or pavement evaluation reports) and maintenance records.

14.2.3 TRAFFIC DATA

The rehabilitation procedures presented in this manual require a characterization of the traffic expected to be carried by the road after rehabilitation. Such a characterization is done in terms of a cumulative number of equivalent standard axles (ESAs). This method of characterization is necessary because the deterioration of paved roads due to traffic results from both the magnitude of the individual wheel loads and the number of times these loads are applied. Equivalency factors are used to convert traffic volumes into cumulative equivalent standard axles loaded at 8.2 metric tonnes.

The process by which the cumulative number of ESAs is determined is illustrated in Figure 2.1, and is essentially as described in the previous chapters. The details of the determination are given in Chapter 3 and Chapter 2, respectively. The user of this rehabilitation manual should refer to these chapters for such details which will not be repeated herein.
Some notes are, nevertheless, worth mentioning, as follows:

- The design period of a rehabilitation project may be subject to constraints of a different nature than those relative to a new construction project. In some cases, for instance, a rehabilitation project may be necessary during a relatively short period, during which an alternate alignment is being designed and/or constructed. The design period should be carefully ascertained with RDA at the outset of the rehabilitation design.

- Vehicle classification, initial traffic volume determination and cumulative traffic volumes over the design period shall be as detailed in the previous chapters of this manual.

- The conversion of traffic volumes into cumulative ESAs over the design period, as detailed in the previous chapters of this manual, is effected through the use of axle load equivalency factors. These equivalency factors could be calculated using standard formulae; they are given in tabular form in the previous chapters of this manual and are
relative to single axle loads. For vehicles with multiple axles, i.e. tandems, triples, etc., each axle in the multiple group is considered separately, and weighted separately in axle load surveys.

- It is noted that the rehabilitation methods presented in this manual are derived from methods proposed by the Asphalt Institute (e.g. Ref. 11), which uses equivalency factors for tandem axles. The use of equivalency factors considering each axle separately, as detailed in the previous chapters of this manual, is comparatively conservative and is recommended.
14.3 ANALYSIS – NEED FOR REHABILITATION

14.3.1 INTRODUCTION

Upon completion of the data collection and fieldwork, the engineer normally has all the elements necessary to perform the rehabilitation design. It is worth, however, evaluating at this point, at the project level, the need for rehabilitation by applying the method of determination used by the PMS mostly at the network level.

If the need for rehabilitation is not confirmed, this may indicate that the most recent data collected for the project has refined the earlier, tentative determination that rehabilitation was needed for a particular project, and that rehabilitation should be reconsidered. It may also indicate, on the other hand, that the project under consideration is of such a nature and/or presents such overriding characteristics that have not yet been entirely incorporated into the overall PMS system, which does not operate on a strictly technical basis.

The purpose of this chapter is to outline the method used in the PMS to differentiate between the needs for rehabilitation or maintenance for flexible pavements. The method is given in detail for reference in Appendix C (copied from Ref. 12).

14.3.2 DAMAGE QUANTIFIERS

During the pavement condition survey (cf. Appendix B.1), two types of damage (A and B) have been noted, together with their extent and severity level. Type B damage generally influences the type of work to be done only if there is no Type A damage. In the method, a global visual index $I_s$ is used, which is based solely on Type A damage (indicative of structural condition).

The global visual index $I_s$ is calculated from three damage groups:

- cracking and crazing
- deformation and rutting
- repairs

The index $I_s$ (from 1 to 7) is calculated following the flow chart given in Figure 3.1. Ratings of 1 and 2 reflect good surface conditions requiring no work or for which work can safely be postponed. Ratings 3 and 4 represent an intermediate surface condition, serious enough to trigger maintenance work in the absence of any other consideration. Ratings 5, 6 and 7 represent very poor surface conditions requiring major maintenance or rehabilitation work.

14.3.3 PAVEMENT QUALITY RATING

A pavement quality rating $Q_i$ is estimated by combining the value of the index $I_s$ as defined above, characterizing the “visual condition”, with the results of deflection measurements qualifying the combined “bearing capacity” of the pavement structure and its subgrade support.
The deflection of an homogeneous section of road is characterized by its value “d” (in $1/100$ mm). In the PMS method, “d” is defined as “$m + 1.3 \sigma$”, “m” being the mean value of the deflection measurements and “$\sigma$” their standard deviation.

### Visual Examination

#### Cracking Index

<table>
<thead>
<tr>
<th>Extent</th>
<th>0 to 10%</th>
<th>10 to 50%</th>
<th>&gt;50%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Severity</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
</tbody>
</table>

#### Deformation Index

<table>
<thead>
<tr>
<th>Extent</th>
<th>0 to 10%</th>
<th>10 to 50%</th>
<th>&gt;50%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Severity</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
</tbody>
</table>

(1) Separate calculation for longitudinal cracking and crazing. The larger of the two indices is used.

### First Damage Index

<table>
<thead>
<tr>
<th>Id</th>
<th>0</th>
<th>1-2</th>
<th>3</th>
<th>4-5</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>1-2</td>
<td>3</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>3</td>
<td>4</td>
<td>5</td>
<td>5</td>
<td>6</td>
</tr>
<tr>
<td>4-5</td>
<td>5</td>
<td>6</td>
<td>7</td>
<td>7</td>
</tr>
</tbody>
</table>

Correction for repairs

### Surface Damage Index

<table>
<thead>
<tr>
<th>Extent</th>
<th>0 to 10%</th>
<th>10 to 50%</th>
<th>&gt;50%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Severity</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>0</td>
<td>+1</td>
</tr>
<tr>
<td>3</td>
<td>0</td>
<td>+1</td>
<td>+1</td>
</tr>
</tbody>
</table>

Figure 3.1 (from Ref. 12) Determination of Damage Index $I_s$
Note: In the PMS study (Ref. 5), a choice has been made to measure (with a Benkelman Beam) the deflections between the twin-wheels of an axle loaded at the legal limit in Somaliland, i.e. 10 metric tonnes. In order to calculate “d” as indicated above, it is suggested that a linear interpolation be used, from the measurements made with an axle loaded at 8.2 metric tonnes as prescribed in Appendix B. Although imperfect, this provides a reasonable consistency to link the design methods presented in this manual with the PMS procedure.

The characteristic deflection “d” having been determined, it is then compared with two threshold values $d_1$ and $d_2$, which divide general deflection magnitudes into three ranges as follows:

- $d_1$ value below which pavement performance is generally good
- $d_2$ value above which pavement performance is poor
- $d_1 - d_2$ range of indecision

The values of $d_1$ and $d_2$ are not necessarily fixed and are subject to revision as the PMS evolves on a network level. They may also be traffic dependent. The current values should be obtained from the PMS Branch.

As an illustration, at the time of the introduction of the PMS, the study selected the “threshold” deflections as:

- For roads with a surface treatment
  - $d_1 = \frac{90}{100}$ mm
  - $d_2 = \frac{115}{100}$ mm

- For roads with surfacings of asphalt concrete of more than 8 to 10 cm
  - $d_1 = \frac{60}{100}$ mm
  - $d_2 = \frac{80}{100}$ mm

The deflections were chosen on the basis of a small number of test sections. The deflection “threshold” should be reviewed from time to time and the designer is expected to verify the current values.

Based on the deflection “d” compared to $d_1$ and $d_2$, the characteristic deflection of a particular road falls into one of three classes, as illustrated in Figure 3.2. Figure 3.2 also defines the overall Pavement Quality Rating $Q_i$ based on the possible combinations of deflection value and Surface Damage Index $I_s$.

Based on the Quality Rating $Q_i$, the method provides the following options, as illustrated in Figure 3.2:

- For quality ratings $Q_1$ to $Q_3$, no major rehabilitation work is required. The road works to be performed include routine maintenance and/or periodic maintenance.
- For quality ratings $Q_4$ to $Q_6$, this indicates a zone of indetermination where visual inspection and deflection values appear inconsistent. A procedure is given in Appendix C to reassess the rating and to reclassify it as either in the range $Q_2$-$Q_3$ or $Q_7$-$Q_8$.
- For quality ratings $Q_7$ to $Q_9$, a rehabilitation is required (e.g. in the form of a structural overlay).
<table>
<thead>
<tr>
<th>Surface damage index Is</th>
<th>Deflection</th>
<th>d1 (maintenance)</th>
<th>d2 (maintenance)</th>
<th>Class 1</th>
<th>Class 2 (to be reclassified)</th>
<th>Class 3 (overlay)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 - 2</td>
<td>Q1</td>
<td>Q3</td>
<td>Q6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Little or no cracking or no deformation</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 - 4</td>
<td>Q2</td>
<td>Q5</td>
<td>Q8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cracks with little or no deformation, deformation without cracks</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5 – 6 - 7</td>
<td>Q4 (to be reclassified)</td>
<td>Q7 (overlay)*</td>
<td>Q9 (overlay)*</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cracks and deformation</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* or other rehabilitation method

Figure 3.2 Pavement Quality Rating ($Q_i$) and Required Road Works (adapted from Ref. 5)
14.4  DESIGN OF ASPHALT OVERLAYS

14.4.1  ASPHALT OVERLAYS OF FLEXIBLE PAVEMENTS

GENERAL

Asphalt overlays may be used to correct both surface deficiencies (raveling, roughness, slipperiness) and structural deficiencies. Surface deficiencies in asphalt pavements usually are corrected by thin resurfacings (functional overlays), but structural deficiencies require overlays designed on factors such as pavement properties and traffic loadings (structural overlays).

There are many instances when a surface treatment will not accomplish what is needed. Examples are depressions or severe raveling. In such cases, a thin overlay should be used over any required leveling course. Thin overlays usually range from 2.5 cm to 5 cm thick using a fine-grained top size dense mix. These are considered maintenance.

The overlay design procedures in the remainder of this section provide an overlay thickness to correct a structural deficiency. If no structural deficiency exists, a thin overlay may still be required to correct a functional deficiency.

This section covers the design of structural overlays by means of one or several lifts of asphalt concrete.

It is assumed that this option is feasible, i.e. that the condition of the existing pavement is not such that it dictates substantial removal and replacement of the existing pavement. Such conditions would include:

- A large amount of very severe alligator cracking
- Excessive rutting which can be attributed to unstable existing materials
- Seriously deteriorated stabilized roadbase requiring an excessive amount of repairs prior to overlay operations
- Contaminated granular roadbase
- Excessive stripping of the existing AC surface

Two methods of overlay design are recommended, namely a deflection procedure (adapted from the Asphalt Institute, Ref. 11) and an effective thickness (or component analysis) procedure (adapted from AASHTO, Ref. 3). It is recommended that both methods always be used for comparison purposes. It is unlikely that the methods will agree exactly, and sound engineering judgment is required to estimate the possible reasons for the discrepancies and make a choice or a compromise between the results obtained by both methods.

Preference may be given to the effective thickness procedure when the history of pavement construction is well known and the destructive testing results are such that the quality of the materials is also well-known. When the results and the records indicate the possibility of significant variation and uncertainty in the structure of the existing pavement, or extensive localized repairs, it is probably preferable to rely on statistical deflection results indicative of the overall load carrying capacity of the system comprising the pavement and its supporting subgrade.
DEFLECTION PROCEDURE

The deflection procedure recommended herein uses the results of a deflection survey conducted with a Benkelman Beam as described in Section B.3.1

The steps involved in the procedure are as follows:

- Step 1: Determine a representative rebound deflection (RRD)
- Step 2: Determine the design future traffic in terms of cumulated equivalent standard axles (ESAs).
- Step 3: Determine the required overlay thickness.

**Step 1**

The individual deflection measurements recorded during the deflection survey must be adjusted by a temperature adjustment factor which can be read from Figure 4.1. The determination of the mean pavement temperature can be made in accordance with the procedure given in Appendix E.

![Figure 4.1](image)

**Figure 4.1 (from Ref. 15): Temperature Adjustment Factor for Benkelman Beam Deflections**

The mean and standard deviation of the adjusted individual deflection readings are then calculated. The representative rebound deflection RRD is taken as:

\[
RRD = (x + 2s)c
\]

Where:
- \(x\) is the arithmetic mean of the individual deflection measurements adjusted for temperature
- \(s\) is the standard deviation of the adjusted individual measurements
- \(c\) is a critical period adjustment factor

---

1 The results of deflection surveys conducted with a FWD are well-suited for use, after back-calculation of pavement parameters, as an input to the effective thickness procedure.
The critical period is the interval during which the pavement is most likely to be damaged by heavy loads. The Asphalt Institute (Ref. 11) recommends the following methods for determining the critical period adjustment factor:

a) Obtain a continuous record of measured rebound values for a similar pavement in a similar environment and on a similar subgrade, and determine the most critical period. Then either:
   1) Make the rebound measurements during the critical period, in which case the adjustment factor, c, equals 1.0. Or:
   2) Make the rebound measurements at any time and adjust to the critical deflection by letting the adjustment factor, c, equal the ratio of the critical period deflection to the deflection for the date of the test.

b) If no record of comparable deflection data is available, make the rebound measurements at any time and make any needed adjustments using engineering judgment.

Example

Rebound deflections were measured for a section of asphalt pavement under uniform temperature conditions. The measurements were made under an 8.2 tonne axle load and the pavement temperature was estimated to be 17° C. The aggregate roadbase of the pavement was 350 mm thick. Using Figure 4.1, the temperature adjustment factor is $F = 1.04$. The measurements were made at the end of the rainy season and may reasonably be considered to be representative of the critical period, hence $c = 1.0$. Individual and adjusted measurements, and calculated representative deflection, are given below:

<table>
<thead>
<tr>
<th>Measured deflections (under 8.2 tonne axle), mm</th>
<th>Deflections adjusted for temperature (factor 1.04), mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.60</td>
<td>0.62</td>
</tr>
<tr>
<td>0.56</td>
<td>0.58</td>
</tr>
<tr>
<td>0.46</td>
<td>0.48</td>
</tr>
<tr>
<td>0.70</td>
<td>0.72</td>
</tr>
<tr>
<td>0.80</td>
<td>0.83</td>
</tr>
<tr>
<td>0.68</td>
<td>0.71</td>
</tr>
<tr>
<td>0.57</td>
<td>0.60</td>
</tr>
<tr>
<td>0.71</td>
<td>0.74</td>
</tr>
<tr>
<td>0.53</td>
<td>0.55</td>
</tr>
<tr>
<td>0.86</td>
<td>0.90</td>
</tr>
</tbody>
</table>

Mean adjusted value (mm) 0.673
Standard deviation (mm) 0.130
Representative deflection (mm) 0.932

Step 2
The traffic analysis outlined in Section 2.3 of this manual is to be used to determine the design number of equivalent standard axles (ESAs) expected to be carried by the road after overlay. As a reminder, this design number of ESAs is expressed in equivalent 8.2 tonne axles.

**Step 3**

To find the thickness of asphalt concrete overlay required, enter the overlay thickness design chart, Figure 4.2, with the RRD obtained in Step 1, move vertically to the curve representing the design ESA (from Step 2), and move horizontally to the Overlay Thickness scale.

![Overlay Thickness Design Chart](image)

**Example**

The pavement considered in the example in Step 1 is being considered for a future traffic of $5 \times 10^6$ ESAs. With a RRD of 0.93 mm, the required overlay thickness is found from the chart to be 75 mm.

**Effective Analysis Procedure**

The required thickness of AC overlay is computed as:

$$T_0 = \frac{(SN_{\text{new}} - SN_{\text{eff}})}{a_0}$$

where

$T_0 =$ required overlay thickness in centimeters  
$SN_{\text{new}} =$ structural number of a new pavement (centimeters)  
$SN_{\text{eff}} =$ effective structural number of the existing pavement (centimeters)  
$a_0 =$ structural coefficient of the AC overlay

It may be noted that the structural numbers have the same dimension as a thickness.
SN\textsubscript{\text{new}} is computed as indicated in Appendix F. It requires the determination of the required structure of a new pavement for the specific subgrade strength and traffic applicable to the project, in accordance with the procedure detailed in RDA’s *Pavement Design Manual* - 2014. The procedure given in Appendix F lists structural layer coefficients for the conversion of the required structure into \( \text{SN}_{\text{new}} \). The same coefficient as given for new AC surface course materials may be used for the structural coefficient \( a_0 \) of the AC overlay.

\( \text{SN}_{\text{eff}} \) requires knowing the existing pavement structure and using the equation:

\[
\text{SN}_{\text{eff}} = a_1 T_1 + a_2 T_2 m_2 + a_3 T_3 m_3
\]

where

\( T_1, T_2, T_3 = \) thicknesses (in centimeters) of existing pavement surface, roadbase, and subbase layers

\( a_1, a_2, a_3 = \) corresponding structural layer coefficients

\( m_2, m_3 = \) drainage coefficients for granular roadbase and subbase

The thicknesses \( T_i \) are determined from the previously collected data and field work, as indicated in Chapter 2 and Appendix B.

The coefficients \( a_i \) may be determined from Table 4.1, which lists suggested layer coefficients for commonly used materials. Other suggested coefficients, for stabilized roadbase materials, are given in Table 4.2.

### Table 4.1 (from Ref. 3)

**Suggested Layer Coefficients for Existing AC Pavement Layer Materials**

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>SURFACE CONDITION</th>
<th>COEFFICIENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC Surface</td>
<td>Little or no alligator cracking and/or only low-severity transverse cracking</td>
<td>0.35 to 0.40</td>
</tr>
<tr>
<td></td>
<td>&lt;10 percent low-severity alligator cracking and/or &lt;5 percent medium- and high-severity transverse cracking</td>
<td>0.25 to 0.35</td>
</tr>
<tr>
<td></td>
<td>&gt;10 percent low-severity alligator cracking and/or &lt;10 percent medium-severity alligator cracking and/or &gt;5-10 percent medium- and high-severity transverse cracking</td>
<td>0.20 to 0.30</td>
</tr>
<tr>
<td></td>
<td>&gt;10 percent medium-severity alligator cracking and/or &lt;10 percent medium-severity alligator cracking and/or &gt;10 percent medium- and high-severity transverse cracking</td>
<td>0.14 to 0.20</td>
</tr>
<tr>
<td></td>
<td>&gt;10 percent high-severity alligator cracking and/or &gt;10 percent high-severity transverse cracking</td>
<td>0.08 to 0.15</td>
</tr>
<tr>
<td>Granular Roadbase or Subbase</td>
<td>No evidence of pumping, degradation, or contamination by fines. Some evidence of pumping, degradation, or contamination by fines.</td>
<td>0.10 to 0.14</td>
</tr>
</tbody>
</table>

### Table 4.2 (from Ref. 3)
Additional Suggested Layer Coefficients for Stabilized Roadbase Materials

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>SURFACE CONDITION</th>
<th>COEFFICIENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stabilized</td>
<td>Little or no alligator cracking and/or only low-severity transverse cracking</td>
<td>0.20 to 0.35</td>
</tr>
<tr>
<td>Roadbase</td>
<td>&lt;10 percent low-severity alligator cracking and/or &lt;5 percent medium- and high-severity transverse cracking</td>
<td>0.15 to 0.25</td>
</tr>
<tr>
<td></td>
<td>&gt;10 percent low-severity alligator cracking and/or &gt;5-10 percent medium- and high-severity transverse cracking</td>
<td>0.15 to 0.20</td>
</tr>
<tr>
<td></td>
<td>&gt;10 percent medium-severity alligator cracking and/or &gt;5-10 percent medium- and high-severity transverse cracking</td>
<td>0.10 to 0.20</td>
</tr>
<tr>
<td></td>
<td>&gt;10 percent high-severity alligator cracking and/or &gt;10 percent high-severity transverse cracking</td>
<td>0.08 to 0.15</td>
</tr>
</tbody>
</table>

It must be realized that relatively limited guidance is available for the selection of layer coefficients for in-service pavement materials. Engineers are encouraged to use judgment and to build experience in the selection of the coefficients, particularly with regard to local materials and pavement behavior.

The drainage coefficients $m_2$ and $m_3$ may be determined on the basis of Table 4.3 and 4.4 further below.

Table 4.3: Quality of Drainage (from Ref. 3)

<table>
<thead>
<tr>
<th>Quality of Drainage</th>
<th>Water Removed Within</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excellent</td>
<td>2 hours</td>
</tr>
<tr>
<td>Good</td>
<td>1 day</td>
</tr>
<tr>
<td>Fair</td>
<td>1 week</td>
</tr>
<tr>
<td>Poor</td>
<td>1 month</td>
</tr>
<tr>
<td>Very Poor</td>
<td>(water will not drain)</td>
</tr>
</tbody>
</table>

Table 4.4 (from Ref. 3)
Recommended $m_i$ Values for Modifying Structural Layer Coefficients of Untreated Roadbase and Subbase Materials

<table>
<thead>
<tr>
<th>Quality of Drainage</th>
<th>Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Less Than 1%</td>
</tr>
<tr>
<td>Excellent</td>
<td>1.40-1.35</td>
</tr>
<tr>
<td>Good</td>
<td>1.35-1.25</td>
</tr>
<tr>
<td>Fair</td>
<td>1.25-1.15</td>
</tr>
<tr>
<td>Poor</td>
<td>1.15-1.05</td>
</tr>
<tr>
<td>Very Poor</td>
<td>1.05-0.95</td>
</tr>
</tbody>
</table>

Example
An existing pavement is made up of the following layers:

- 5 cm AC surfacing ($T_1$)
- 15 cm granular roadbase ($T_2$)
- 15 cm subbase ($T_3$)

The AC surface shows less than 10 percent of low-severity cracking, very little medium- and high-severity transverse cracking, and can be assigned (cf. Table 4.1) a structural layer coefficient $a_1$ of 0.30. Roadbase and subbase courses show no evidence of degradation or contamination. The coefficients $a_2$ and $a_3$ may both be taken as 0.12. The quality of drainage is considered fair and the pavement structure is exposed to levels near saturation on the order of 5%. Both coefficients $m_2$ and $m_3$ can be taken as 1.00.

$SN_{\text{eff}}$ is calculated as:

$$SN_{\text{eff}} = 0.30 \times 5 + 0.12 \times 1.00 \times (15 + 15) = 5.1$$

It is contemplated to overlay the pavement for an expected traffic class T6. The subgrade strength class is S4. The structural number of an adequate pavement structure for these conditions is $SN_{\text{new}} = 9.13$ (see Appendix F).

The required overlay thickness is: $T_0 = (9.13 - 5.1)/0.44 = 9.15$ cm (which may be rounded up to 10 cm).

Note: The above method is based on the determination of $SN_{\text{eff}}$ essentially from an assessment of the quality of the pavement layers from visual, field (e.g., DCP) and/or laboratory testing. Alternatively, $SN_{\text{eff}}$ may be estimated from the results of deflection (FWD) testing as outlined in Appendix G.

**Surface Preparation for Overlay**

In the design of overlays and the adoption of the overlay as a rehabilitation solution, the construction feasibility should be verified first (besides the economic constraints) with reference to factors such as:

- Traffic control, traffic disruption
- Materials and equipment availability
- Construction problems such as utilities, bridge clearances, side slope extension

Having determined the feasibility, careful and correct preparation of the existing pavement, prior to construction of overlays, is essential to good construction and to maximum overlay performance. The overlay thickness is designed to correct a below-average pavement condition, but not to provide the extra structural strength needed for localized weak areas. If the overlay thickness is based on the weakest condition in the section, it would be over-designed for the rest of the section and thus be needlessly costly. Therefore, the weaker areas must be corrected to provide a uniform foundation for the overlay.

Some of the factors which need consideration in preparation of the existing pavement are as follows:

**Pre-Overlay Pavement Repairs**
If distress in the existing pavement is likely to affect the performance of the overlay, it should be repaired prior to the placement of the overlay. Much of the deterioration that occurs in overlays results from deterioration that was not repaired in the existing pavements. The cost tradeoffs of pre-overlay repair and overlay type should also be considered.

Severe alligator cracking and linear cracks, rutting and surface irregularities should be repaired prior to overlay of AC pavements.

The pre-overlay repairs generally fall in the maintenance categories. One particular pre-overlay operation to consider is an effective reflection crack control.

**Reflection Crack Control**

Reflection cracks are a frequent cause of overlay deterioration. The thickness design procedures described in the preceding sections do not consider reflection cracking. Pre-overlay repairs (patching and crack filling) may help delay the occurrence and deterioration of reflection cracks. Additional reflection crack control measures include:

- Pavement fabrics
- Crack relief layers. These are composed of open-graded coarse aggregate and a small percentage of asphalt cement.
- Increased overlay thickness

**Subdrainage**

As indicated in Appendix B, the existing subdrainage condition of the pavement should be evaluated since it has a great influence on how well the overlay performs. Removal of excess water from the pavement cross-section will increase the strength of the pavement layers and subgrade, and reduce deflections.

**Milling-Recycling**

Milling, with or without the intent of recycling the milled materials, can improve the performance of the overlay, by removing some of the cracked and hardened materials and by minimizing existing rutting or other significant distortions.

**Surface Recycling**

This process may be considered as analogous to pre-overlay surface preparation or an in-place variant of cold milling and recycling. The asphalt pavement surface is heated in place, scarified, remixed, relaid, and rolled. Asphalts, recycling agents, new asphalt hot-mix, aggregates, or a combination of these may be added to obtain a desirable mixture. When new asphalt hot-mix is added, the finished product may be used as the final surface; otherwise, an asphalt surface course should be used.

**Shoulders**

Overlaying traffic lanes generally requires that the shoulders be overlaid to match the grade line of the traffic lanes. In selecting an overlay material and thickness for the shoulder, the designer should consider the extent to which the existing shoulder is deteriorated and the amount of traffic that will use the shoulder. For example, if trucks
tend to park on the shoulder at certain locations, this should be considered in the shoulder overlay design.

If an existing shoulder is in good condition, any deteriorated areas should be patched. An overlay may then be placed to match the shoulder grade to that of the traffic lanes. If an existing shoulder is in such poor condition that it cannot be patched economically, it should be removed and replaced.

14.4.2 ASPHALT OVERLAYS OF RIGID PAVEMENTS

DEFLECTION PROCEDURE

Deflection tests measure the response of a pavement to a specified load. Depending on the type of pavement, the measured deflection indicates the necessity for overlay. The thickness of the overlay can be reduced by limiting the deflections by way of undersealing.

In Jointed Plain Concrete Pavements (JPCP), the differential deflection should be $\leq 0.05$ mm, and mean edge deflection should be $\leq 0.36$ mm. In Continuously Reinforced Concrete Pavement (CRCP), the deflection should be less than 0.015 mm. If the above criterion is exceeded, overlaying with or without undersealing is required. The deflections are assumed to be reduced by 0.2 percent per millimeter of overlay thickness. The thickness of overlay can be obtained from the chart shown in Figure 4.3, based on the temperature differential and the length of the slab. Alternatives 2 and 3, mentioned in the figure, can be used to reduce the overlay thickness in sections B and C of the chart.

Example

Slab = 13.5 m Jointed Plain Concrete Pavement (JPCP)

High Temperature = 28° C

Low Temperature = -16° C

$\Delta t = 44° C$

Measured Benkelman Beam deflections:

$d_1 = 0.85$ mm

$d_2 = 0.63$ mm

Diff. Deflection = $d_1 - d_2 = 0.85 - 0.63 = 0.22$ mm

Mean Deflection = $(d_1 + d_2)/2 = (0.85 + 0.63)/2 = 0.74$ mm
NOTES:
(1) Temperature differential (Δt) is the difference between the highest normal daily maximum temperature and the lowest normal daily maximum temperature for the hottest and coldest months, based on a 30-year average.
(2) Alternative 2: Crack and seat the slab into smaller sections with thinner overlay.
(3) Alternative 3: Utilize a crack-relief layer with drainage system with thinner overlay.

Figure 4.3 (from Ref. 11)  Chart for Selecting Asphalt Concrete Structural Overlay Thickness for PCC Pavement

**Alternative 1: Thick Overlay**

Step 1—Enter the design chart, Figure 4.3, with Δt = 44°C and slab length = 13.5 m
The required overlay is more than 230 mm. Therefore, use Alternative 2 or 3.

Alternative 2: Reduce Slab Length

Step 1—Break the slab into 6.8 m sections.

Step 2—Enter the design chart, Figure 4.3, with $\Delta t = 44^\circ$ C and slab length = 6.8 m

Required overlay = 160 mm (by interpolation)
Use 165 mm

Step 3—Check vertical mean deflection:
At the rate of 0.2 percent per millimeter of overlay thickness, the reduction in deflection is: $165 \times 0.74 \times 0.002 = 0.24$ mm
Remaining deflection is: $0.74 - 0.24 = 0.50$ mm $> 0.36$ mm allowable.
Therefore, undersealing is required.

**Effective Thickness Procedure**

If the overlay is being placed for some functional purpose such as roughness or friction, a minimum thickness overlay that solves the functional problem should be placed. If the overlay is being placed for the purpose of structural improvement, the required thickness of the overlay is a function of the structural capacity required to meet future traffic demands and the structural capacity of the existing pavement. The required overlay thickness to increase structural capacity to carry future traffic is determined by the following equation:

$$D_{ol} = A(D_f - D_{eff})$$

where

- $D_{ol}$ = Required thickness of AC overlay, cm
- $A$ = Factor to convert PCC thickness deficiency to AC overlay thickness
- $D_f$ = Slab thickness to carry future traffic, cm
- $D_{eff}$ = Effective thickness of existing slab, cm

The $A$ factor, which is a function of the PCC thickness deficiency, is given by the following equation:

$$A = 2.2233 + 0.0015(D_f - D_{eff})^2 - 0.0604(D_f - D_{eff})$$

AC overlays of conventional JPCP, JRCP, and CRCP have been constructed as thin as 5 cm and as thick as 25 cm. The most typical thicknesses that have been constructed for highways are 7.5 to 15 cm.

$D_{eff}$ is computed from the following equation:

$$D_{eff} = F_{jc} \times F_{dur} \times F_{fat} \times D$$

where
The factors $F_{jc}$, $F_{dur}$, $F_{fat}$, are dependent on the existing condition of the pavement and can be evaluated based on the condition survey detailed in Appendix B.

$F_{jc}$ depends on the total number of unrepaired deteriorated joints, cracks, punchouts and other discontinuities per kilometer in the design lane and is determined using Figure 4.4.

$F_{dur}$ depends on the existing durability problems, such as aggregate distress. Using the condition survey, $F_{dur}$ is determined as follows:

- 1.00: No sign of PCC durability problems
- 0.96-0.99: Durability cracking exists, but no spalling
- 0.88-0.95: Substantial cracking and some spalling exists
- 0.80-0.88: Extensive cracking and severe spalling exists

The $F_{fat}$ factor adjusts for past fatigue damage that may exist in the slab. It is determined by observing the extent of transverse cracking (JPCP, JRCP) or punchouts (CRCP) that may be caused primarily by repeated loading.

The following guidelines can be used to estimate the $F_{fat}$ factor in the design lane:

- 0.97-1.00: Few transverse cracks/punchouts exist (none caused by “D” cracking or reactive aggregate distress)
  - JPCP: <5 percent slabs are cracked
  - JRCP: <25 working cracks per mile (about 16 per km)
  - CRCP: <4 punchouts per mile (2 or 3 per km)
- 0.94-0.96: A significant number of transverse cracks/punchouts exist (none caused by “D” cracking or reactive aggregate distress)
  - JPCP: 5-15 percent slabs are cracked
  - JRCP: 25-75 working cracks per mile (16-47 per km)
  - CRCP: 4-12 punchouts per mile (3 to 8 per km)
- 0.90-0.93: A large number of transverse cracks/punchouts exist (none caused by “D” cracking or reactive aggregate distress)
  - JPCP: >15 percent slabs are cracked
  - JRCP: >75 working cracks per mile (>47 per km)
  - CRCP: >12 punchouts per mile (>8 per km)
Deteriorated Transverse Joints and Cracks / Kilometer

Figure 4.4 \( F_K \) Adjustment Factor

**SURFACE PREPARATION FOR OVERLAYS**

The following types of distress in JPCP, JRCP and CRCP should be repaired prior to placement of an AC overlay.

<table>
<thead>
<tr>
<th>Distress Type</th>
<th>Repair Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Working cracks</td>
<td>Full-depth repair or slab replacement</td>
</tr>
<tr>
<td>Punchouts</td>
<td>Full-depth PCC repair</td>
</tr>
<tr>
<td>Spalled joints</td>
<td>Full-depth or partial-depth repair</td>
</tr>
<tr>
<td>Deteriorated repairs</td>
<td>Full-depth repair</td>
</tr>
<tr>
<td>Pumping/faulting</td>
<td>Edge drains</td>
</tr>
<tr>
<td>Settlements/heaves</td>
<td>AC level-up, slab jacking, or localized reconstruction</td>
</tr>
</tbody>
</table>

Full depth repairs and slab replacements in JPCP and JRCP should be PCC, dowelled or tied to provide load transfer across repair joints. Full depth repairs in CRCP should be PCC and should be continuously reinforced with steel which is tied or welded to reinforcing steel in the existing slab to provide load transfer across joints and slab continuity.

Installation of edge drains, maintenance of existing edge drains, or other subdrainage improvement should be done prior to placement of the overlay if a sub-drainage evaluation indicates a need for such an improvement.
Pressure relief joints should be placed only at fixed structures, and not at regular intervals along the pavement. The only exception to this is where reactive aggregate has caused expansion of the slab.

Reflection Crack Control

The basic mechanism of reflection cracking is strain concentration in the overlay due to movement in the vicinity of joints and cracks in the existing pavement.

A variety of reflection crack control measures have been used in attempts to control the rates of reflection crack occurrence and deterioration. Any one of the following treatments may be employed in an effort to control reflection cracking in an AC overlay of JPCP or JRCP:

1. Sawing and sealing joints in the AC overlay at locations coinciding with joints in the underlying JPCP or JRCP.
2. Increasing AC overlay thickness. Reflection cracks will take more time to propagate through a thicker overlay and deteriorate more slowly.
3. Placing a bituminous-stabilized granular interlayer (large-sized crushed stone crack relief layer), prior to or in combination with placement of the AC overlay has been effective. See Figure 4.5.
4. Cracking and seating JPCP or breaking and seating JRCP prior to placement of the AC overlay. This technique reduces the size of PCC pieces and seats them in the underlying base, which reduces horizontal (and possibly vertical) movements at cracks.

Reflection cracking can have a considerable (often controlling) influence on the life of an AC overlay of JPCP or JRCP. Deteriorated reflection cracks detract from a pavement’s serviceability and also require frequent maintenance, such as sealing, milling, and patching.

Figure 4.5 (from Ref.11) Crack-Relief Layer in an Overlay System, Cross-Section

When the pavement has been rendered as uniformly stable as possible, it must be cleaned thoroughly and tack-coated with asphalt before the overlay is placed.

When overlaying a PCC pavement that has been grooved, special treatment is necessary to prevent moisture intrusion. Here, a heavy asphalt tack coat, an asphalt slurry seal, or a sand (fine-graded) asphalt leveling course is used to fill the grooves.

Old PCC surfaces that range from polished to coarse-textured can vary significantly in bonding ability. Polished surfaces can be re-textured to improve their bonding with
overlays. However, in most cases, the proper use of a tack coat, selection of the proper mix type and overlay thickness, coupled with correct construction procedures, will prove more economical in ensuring a good bond.
14.5 DESIGN OF ALTERNATIVE REHABILITATION METHODS

Alternative rehabilitation methods other than overlays include, as primary broad categories:

- Reconstruction: In this category, little or no contribution is expected from the existing pavement materials and the materials needed for rehabilitation will be new materials.

- Recycling: The rehabilitation takes advantage of the existing pavement materials, which are reused in part or as a whole in the construction of the rehabilitated pavement.

Other methods which are effective in both repairing existing deterioration and, to some extent, preventing or delaying its recurrence include repairs by patching, crack sealing, milling and surface treatments or seals. These methods, however, are better classified under the term of maintenance (cf. Chapter 1). It is recognized that they are often required for the preparation of the existing pavement prior to overlay (cf. Chapter 4).

The above categories are general in nature, and some combinations of methods are possible and indeed used. For instance, the materials of an existing surface treatment and base course may be recycled as the base or subbase of a reconstructed pavement, with new materials brought on site to construct the upper layers, e.g. roadbase and AC surface.

15.5.1 RECONSTRUCTION

ASPHALT PAVEMENT

Since the pavement is to be reconstructed with new materials, generally only the subgrade of the existing pavement will remain. It is assumed in this case that the existing pavement materials are trucked away. If the materials are removed from the roadway and carried to a central plant for processing and later reuse, this central plant recycling is considered a cold recycling technique.

For reconstruction design, therefore, the subgrade strength (CBR) must be evaluated. This evaluation yields the design CBR to use in pavement design.

The rest of the design process for reconstruction follows that of a new pavement design. This design is to be carried out in accordance with the RDA Pavement Design Manual – 2014 (Ref. 1).

In case a variant reconstruction procedure is used, whereby, for instance, the existing subbase is left in place, prior to the addition of a new roadbase and surface course, the principles of component analysis, as detailed for the overlay design in Chapter 4, may be applied.

Example

An existing pavement is made up of the following layers:
Surface dressing
20 cm of granular roadbase
15 cm of subbase

The condition survey has indicated the need for rehabilitation. The subgrade has been evaluated and is considered properly compacted and homogeneous. Investigations have revealed that the subgrade can be characterized by a CBR within the range of subgrade strength class S4 (CBR between 8 and 14). Traffic studies concluded that the future traffic would be in excess of $6 \times 10^6$ ESA but well within the range of traffic class T6 ($6$ to $10 \times 10^6$ ESA).

An adequate new pavement structure (cf. RDA Pavement Design Manual - 2014) consists of:

- 10 cm AC surfacing
- 20 cm granular roadbase
- 17.5 cm subbase

The existing subbase course is of good quality, with a CBR consistently in excess of 30. The existing roadbase is of marginal or poor quality and new sources of adequate quality of crushed stone have been located. It was elected to reconstruct the pavement from the existing subbase (to be preserved) up. The scarified surface dressing and existing base course will be disposed of.

The existing subbase may remain and contribute to the new pavement structure as 15 cm of subbase (at a 1-to-1 ratio with new materials). It is simpler, in view of the abundance of materials, to substitute crushed stone base materials to the missing 2.5 cm of subbase. The reconstruction operations will consequently lay down 22.5 cm of new roadbase and 10 cm of new AC surfacing.

**PCC PAVEMENT**

At some point near the end of the life of a PCC pavement, the slab is so badly deteriorated that total reconstruction becomes more cost-effective than resurfacing or restoration. PCC pavement conditions that favor reconstruction include:

1. Little or no remaining structural life, as evidenced by extensive slab cracking.
2. Extensive slab settlement, heave, or cracking due to foundation movement (caused by swelling soil or frost heave).
3. Extensive joint deterioration (particularly for short-jointed pavements, since full-depth repair would require replacement of a large percentage of the concrete surface).
4. Extensive concrete deterioration due to poor durability (D-cracking or reactive aggregate distress).

Reconstruction may also be warranted for highways that do not meet required geometric design standards (e.g. lane width, bridge clearance, curve superelevation). On resurfacing
projects, it may sometimes be necessary to reconstruct short sections of pavement near bridges to maintain required bridge clearances.

As is the case with asphalt pavement, in reconstruction projects, generally only the subgrade of the existing pavement will remain. For reconstruction design, therefore, the subgrade strength (CBR) must be evaluated and the rest of the design is to be carried out in accordance with the RDA Pavement Design Manual – 2014 (Ref. 2).

14.5.2 RECYCLING

As long as virgin (new) materials are in abundant supply and within reasonable distance, reconstruction can remain an attractive option. However, when new materials are less plentiful and the costs of haulage become uneconomical, the need to conserve energy and materials favors pavement recycling.

The processes for pavement recycling are conveniently grouped into two categories:

(a) Asphalt Pavement Recycling

This category is further divided into three groups:

- Surface recycling. This type of recycling is described earlier in Section 4.1.
- Cold-mix recycling
- Hot-mix recycling

(b) Portland Cement Concrete Pavement Recycling

These categories are illustrated by Figure 5.1.

![Figure 5.1 Categorization of Recycling Approaches (from Ref. 16)](image)

**RECYCLING OF ASPHALT PAVEMENT**

**Cold-Mix Recycling**
In this process, reclaimed asphalt pavement materials, reclaimed aggregate materials, or both, are combined with new asphalt, and/or recycling agents in place, or at a central plant, to produce cold-mix roadbase mixtures. An asphalt surface course is required. This is a common method for recycling old pavements, including both the surface and untreated roadbase materials.

From the viewpoint of thickness design, the design procedure is similar to that of a reconstruction. The required structure of a new pavement must be determined in accordance with the *RDA Pavement Design Manual* – 2014. A component analysis helps to determine the contribution of the recycled materials.

**Example**

The same example is used as given above for reconstruction, with the following modification: in this case, the subbase is still left in place, but the existing roadbase and surface treatment are being recycled as a bituminous roadbase. The materials are recycled in-place and may be attributed a structural coefficient of 0.25 (see Appendix F).

Keeping the same 1:1 ratio between the existing 15 cm of subbase to be left in place and 15 cm of the required subbase, the deficiency to be made up to complete the required structure is:

- 10 cm of AC surfacing
- 20 cm of granular roadbase
- 2.5 cm of subbase

Using structural layer coefficients of 0.44, 0.14 and 0.11, respectively, this corresponds to a deficit of structural number of 7.5.

One solution consists therefore of laying down 12.5 cm of recycled roadbase (contributing 12.5x0.25 i.e. 3.1 to the SN) and the originally required 10 cm of AC surfacing (contributing 10x0.44 i.e. 4.4 to the SN).

However, alternate solutions providing the same SN may be considered (e.g. 17 cm of recycled roadbase and 7.5 cm of AC surfacing). This illustrates a commonly advocated advantage of recycling.

Cold-mix recycling involves ripping, scarifying, pulverizing, or crushing the old pavement. It is either hauled to a central site and upgraded with asphalt cement, emulsified asphalt or other stabilizing agents (lime, cement, fly ash in combination with lime or cement, or calcium chloride), or it is treated in place. Treatment in place may be achieved by blading, rotary mixers or a travel plant. Before beginning the project, representative samples should be obtained. The asphalt content and consistency and the gradation of the aggregates should be determined. Then the materials should be blended and a mix design developed. The real economical advantage of cold-mix recycling often lies in the ability to upgrade an old pavement structure with a relatively thin wearing course with limited need for new aggregates. After the recycled mix is placed, it should be capped with an AC wearing course or a surface treatment.

**Hot-Mix Recycling**
In this process, reclaimed asphalt pavement materials, reclaimed aggregate materials, or both, are combined with new asphalt and/or recycling agents and/or new aggregate, as necessary, in a central plant to produce hot-mix paving mixtures.

Two methods are used to process the old pavement: (1) Scarifying and removing the material to a crushing plant where it is sized, or (2) sizing the material in-place with rotating drummilling equipment or hammermills and hauling the sized materials to a central location for later processing. The reclaimed asphalt pavement contains both asphalt and aggregate. Testing is conducted to determine the characteristics of the reclaimed asphalt pavement (e.g. asphalt extraction, gradation of the aggregate and consistency of the extracted asphalt).

Equipment to centrally hot-process the recycled material can be, for convenience, separated into three general categories: direct flame heating, indirect flame heating, and superheated aggregate. The techniques and related operations are illustrated in Figure 5.2.

From the viewpoint of thickness design, the design procedure is identical to that given for cold-mix recycling. It should only be noted that hot-mix recycling lends itself to higher quality control, hence a higher layer structural coefficient may be assigned to the recycled materials.

**RECYCLING OF PORTLAND CEMENT CONCRETE**

In most cases where reconstruction is justified, the existing concrete can be recycled to reduce the cost of reconstruction.

Concrete pavement recycling involves breaking up the old pavement on grade, loading and hauling the material to a crushing plant, and processing it at the plant to produce recycled concrete aggregate (RCA). The product of this process is an aggregate that can be used in place of virgin aggregate in any component of the pavement structure where aggregate is used, including:

1. Untreated, dense-graded aggregate roadbase
2. Cement- and asphalt-stabilized roadbases
3. Lean concrete roadbase
4. Portland cement concrete surfacing
5. Asphalt concrete surfacing
6. Fill
7. Filter material
8. Drainage layer or edge drains
Figure 5.2 (from Ref. 16) Central Plant Hot-Mix Recycling Techniques

* Binder may include with or without modifier. New aggregate may be used as required to correct mix design.
Recycling of PCC pavements is not limited to those pavements that contain sound aggregate. Pavements containing reactive aggregate can be recycled into new concrete using fly ash to control expansion of the reactive aggregate. Badly D-cracked pavements have also been successfully recycled into new pavements. The common practice in recycling D-cracked pavement is to limit the maximum size of the recycled concrete aggregate, so that any aggregate particles that contain sizable voids are fractured before being used in the new concrete mix.

One limitation of small aggregate size is poor aggregate interlock at joints and cracks, especially on JRCP and CRCP. Additional large virgin aggregate may need to be added so that sufficient aggregate interlock will develop.

A PCC pavement that has an AC overlay can be recycled, but the two layers must be recycled separately. Asphalt concrete should not be recycled for aggregate in a PCC mix, because the asphalt cement will inhibit entrainment of air in the concrete mix.

Whether or not a particular project should be recycled depends on numerous factors, including suitability of the pavement for recycling, availability and cost of virgin aggregate, cost of disposing of old pavement material if it is not recycled, approximate cost of recycling, policy toward recycling, and the extent of local contractors’ experience with recycling. PCC pavement recycling has proven to be both economical and environmentally advantageous. Laboratory and field studies have shown that a high quality concrete, with improved durability, can be produced using recycled concrete aggregate. Significant savings in material transportation and disposal costs are possible through recycling, particularly in urban areas.

PCC pavement recycling can be divided into two groups:

- surface recycling
- central plant recycling

Both surface and central plant recycling techniques have been utilized on roadways containing Portland cement concrete. However, surface recycling techniques applied to concrete pavements is most often considered a pavement removal operation. The removed pavement can be recycled. Figure 5.3 describes some of the Portland cement concrete recycling options.

**Surface Recycling**

Surface recycling techniques involve the use of cold milling or cold planing techniques that are capable of economically removing up to approximately two inches of concrete in a single pass. Traffic can operate for extended periods of time on the milled surface, or an asphalt concrete overlay can be placed.

The sequence of operation involving surface recycling is as follows:

1. Establishing desirable grade line.
2. Milling, grinding, or planing the pavement to the desired depth.
3. Clean-up involving rotary broom and vacuum equipment.
4. Disposal or recycling of the millings.
Surface recycling operations, involving overlays, use the sequence of operations as described above, with the addition of a tack coat and asphalt concrete overlay.

Pavement milling operations are suitable for the removal of localized severe surface undulations caused by swelling clays, etc.; for removal of pavement prior to overlay along gutters, at bridge approaches, and other areas where a feathered edge of asphalt concrete or Portland cement concrete is likely to abrade; and for improved drainage, surface texture, and skid resistance.

An added advantage of surface recycling is the increase in bond strength between a milled Portland cement concrete and an overlay as compared to a normal overlay operation. This anticipated increase in bond strength may allow the use of thin overlays on Portland cement concrete pavements.

**Central Plant Recycling**

The pavement removal and crushing operations are performed with conventional construction and demolition equipment or specially designed equipment. The old concrete pavement is normally broken such that the size of the resulting slab is normally small enough to be received by the primary crusher. Additional reduction in slab size can be performed at the crushing location. Central plant sizing can be performed with conventional, fixed and portable crushing equipment; however, reinforcing steel may be a problem and may have to be removed at one or more of six processing locations:

1. On the grade during the loading operation,
2. During the locating operation for crushing if stockpiling occurs prior to crushing,
3. At the entry to the primary crushing,
4. On the belt after primary crushing,
5. On the belt after final crushing, or
6. In the stockpile prior to remixing.

Equipment recently developed pulverizes the concrete to smaller sizes on grade and thus more complete steel removal is possible on grade.

Blending and mixing operations in the central plant are standard operations, as are the techniques utilized for the placing and curing of recycled materials. Gradation adjustments (particularly the addition of natural sands) are often made to improve workability.

**Structural Design**

The structural design of PCC pavement using recycled materials can be done, in principle, using the guidelines given in the *RDA Pavement Design Manual, - 2014* (Ref. 2).

In essence, recycled material is viewed no differently from new or virgin material in either new or rehabilitated pavement systems. Because of this, the most important structural rehabilitation parameter for recycled material is the characterization of the load carrying ability (lab and/or field) of the material.
Break Pavement \rightarrow Load and Haul to Plant \rightarrow Crush, screen and stockpile

- Blend crushed PCC with new aggregate as required.
- Blend crushed PCC with new aggregate as required and add Portland cement or asphalt.
- Blend crushed PCC with new aggregate as required and add Portland cement and water to make lean concrete - econocrete.
- Blend crushed PCC with new aggregate as required and add Portland cement and water to make Portland cement concrete.
- Blend crushed PCC with new aggregate as required and add asphalt cement to make asphalt concrete.

Haul, place and cure

Figure 5.3 (from Ref. 16)  Portland Cement Concrete Pavement Recycling
Literature review indicates that the structural coefficients of recycled materials do reflect the extreme variation of reused materials and recycling processes utilized. However, it is equally apparent that recycled materials have the capacity to be equivalent, or greater, in load spreading capabilities than the original material. With good control on the recycling process, and based on laboratory test data, an appropriate structural coefficient can be evaluated to be used in the design.

Similarly, the properties of concrete made with recycled aggregate show some variations compared to the concrete made with virgin aggregate, as given below.

**Concrete Properties**

The following is a comparison of the properties of PCC made with recycled concrete aggregate and concrete made with virgin aggregate:

1. The compressive strength of recycled concrete is between 60 and 100 percent of the compressive strength of conventional concrete at the same water-cement ratio. Water-reducing admixtures can be used to obtain higher strength concrete at the same cement content without compromising workability.

2. The static modulus of elasticity of recycled concrete is between 60 and 100 percent of the modulus of conventional concrete at the same water-cement ratio.

3. The flexural strength of recycled concrete is between 80 and 100 percent of the flexural strength of conventional concrete at the same water-cement ratio.

4. Recycled concrete has a higher ratio of flexural strength to compressive strength than conventional concrete.

5. Low-strength recycled concrete can be recycled into higher-strength concrete through proper mix design.

6. The durability of concrete made with aggregate susceptible to D-cracking can be substantially increased by limiting the top size of the aggregate.

7. The volume response to moisture and temperature changes of recycled concrete is not significantly different from that of normal concrete.

The fact that concrete containing recycled concrete aggregate has a higher ratio of flexural strength to compressive strength than conventional concrete should be considered if compressive strength is to be used as the criterion for acceptance or for timing the early opening of reconstruction projects.

All of the above comments about recycled concrete aggregate’s performance in concrete surfaces applies to its use in lean concrete roadbase courses, as well. Recycled concrete aggregate was used in lean concrete roadbases long before it was used in concrete surfaces because not enough was known about its performance in surface courses. In general, the use of recycled concrete aggregate is restricted to roadbase courses (granular, cement- or asphalt-stabilized, and lean concrete).
14.6 SELECTION OF PREFERRED ALTERNATIVE

The following excerpt from Part III of the AASHTO Guide (Ref. 3) appears relevant to this section:

"While analytical solutions to portions of the rehabilitation methodology are presented, the engineer must recognize that it may be impossible to accurately determine the optimal rehabilitation solution from a rigorous analytical model. However, the user should not be discouraged from employing this approach but rather feel encouraged to use every available tool at his/her disposal to determine the problem cause, identify potentially sound and economic solution alternatives, and then select the most preferred rehabilitation strategy from sound engineering experience”.

14.6.1 REHABILITATION FACTORS

Rehabilitation choices should be viewed with reference to several factors, including:

- the decision to use new materials, recycled materials, or a combination of both (noting that recycled materials need not be those obtained from the specific pavement project being rehabilitated)
- the decision to employ full reconstruction, partial reconstruction, a direct overlay, or some combination of reconstruction and overlay.

Due to state of the art limitations regarding the entire rehabilitation process, there is a definite need for feedback on the performance of various rehabilitation methods. The “optimum” solution (from the viewpoint of cost to benefit ratio) may not be attainable for a particular project due to constraints imposed (e.g. funding). There should however be a “preferred” solution which is cost-effective, has other desirable characteristics, and meets the existing constraints.

Some factors which may influence the rehabilitation choices have been listed as data which must be collected prior to the condition survey (cf. Chapter 2). Road inventory is one of them, and the geometric design (i.e. the adequacy of the existing geometry) should be viewed as one major factor to decide between overlay and non-overlay methods. Other potential constraints to be contended with include:

- traffic control problems
- possible funding limitations
- desirable service life of the rehabilitation
- right-of-way
- availability of materials and equipment
- probable contractor’s capacities

On the basis of the above factors and constraints, the design engineer should be able to suggest candidate solutions among those described in Chapters 4 and 5, and among the multiple combinations thereof. After the feasible candidate solutions have been selected, by weighing candidate solutions against project constraints, preliminary design should be prepared. Preliminary designs require only approximate cost estimates.
14.6.2 SELECTION OF PREFERRED SOLUTION

There is no infallible method for selecting the most “preferred” rehabilitation alternative for a given project. The selection process requires engineering judgment, creativity and flexibility. Guidance can nevertheless be offered to select a preferred solution. Both monetary and non-monetary considerations apply.

COST ANALYSIS

The cost is generally considered the most important criterion in the selection process.

Normally, a life-cycle cost analysis is required, and an outline of its main components is given below. However, one must bear in mind a few points regarding life cycle cost analysis as it is applied to the selection of a rehabilitation method.

- Life-cycle cost analysis requires inputs of both cost and time. Unfortunately, both of these elements are subject to a large degree of uncertainty. For instance, the effective life of a rehabilitation technique is subject to the following influences:
  - the skill and care with which the work is performed
  - the quality of the materials used
  - environmental conditions prevalent in the region where the pavement exists
  - the traffic which uses the pavement
  - other rehabilitation and maintenance work being performed concurrently

Even the engineer familiar with the performance of various rehabilitation methods in his or her local area can appreciate the difficulty of selecting appropriate inputs for use in the life-cycle cost analysis. To eliminate as much uncertainty as possible, it is essential to collect rehabilitation performance data whenever available (e.g. PMS Branch).

- Another important consideration regarding life-cycle cost analysis is that the same rehabilitation techniques, when applied to different pavements, may have variant effects. Furthermore, some methods keep a pavement at a consistently high-condition level, while others may allow the condition of the same pavement to fluctuate. Thus, discrepancy is often not revealed by the cost analysis if user costs are not included in the calculations. It is therefore important to include user costs in a cost analysis.

The major costs to consider in the economic analysis include:

(1) Government costs
   (a) Initial rehabilitation costs. These should include, in particular, the costs of pavement preparation (repairs, etc.) required prior to overlay, if this is the alternative considered in the analysis.
   (b) Future rehabilitation costs (after the selected design period for the rehabilitation design).
   (c) Maintenance costs, recurring throughout the design period.
   (d) Salvage return or residual value at the end of the design period.
   (e) Engineering and administrative costs.
(f) Traffic control costs, if applicable.

(2) User costs
   (a) Travel time
   (b) Vehicle operation
   (c) Accidents
   (d) Discomfort
   (e) Time delay and extra vehicle operating costs during resurfacing or major maintenance

There are a number of methods of economic analysis that are applicable to the evaluation of alternative strategies.

(1) Equivalent uniform annual cost method, often simply termed the “annual cost method”
(2) Present worth method for:
   (a) costs
   (b) benefits, or
   (c) benefits minus costs, usually termed the “net present worth” or “net present value method”
(3) Rate-of-return method
(4) Benefit-cost ratio method
(5) Cost-effectiveness method

Either the net present worth value or the equivalent uniform annual cost may be used to determine life cycle costs for comparisons of alternate pavement rehabilitation strategies. In either case, it is essential that comparisons only be made for analysis periods of equal length. Details of the equations used in the methods are beyond the scope of this manual. They may be found in the HDM models developed by the World Bank.

NON-MONETARY CONSIDERATIONS

As with the review of the constraints in the selection of candidate rehabilitation methods, several factors need to be considered in selecting the preferred solution. They include, in particular, service life, duration of construction, reliability of the solution, constructibility and maintainability.

Also, as with monetary considerations, the service life (selected and ascertained during the data collection) of a rehabilitation method is an important factor. This is particularly significant for high-volume roads, for which lane closures and traffic delays pose considerable difficulties. The important time parameter is years of pavement life extension achieved by the rehabilitation methods and should be a factor in almost any decision criterion.

PREFERRED REHABILITATION ALTERNATIVE

The preferred rehabilitation alternative will be selected using first cost considerations, then monetary factors. When the cost does not indicate a clear advantage, weighing factors may be assigned to the non-monetary considerations. Each factor is evaluated and
multiplied by its weight, and a final “score” is calculated. This procedure is in relatively common use to select the preferred rehabilitation alternative.

Such a method for measuring several rehabilitation alternatives against criteria that cannot be expressed in monetary units is illustrated in Figure 6.1.

Detailed design, plans and cost estimates are normally prepared after the selection of the preferred alternative. If major differences occur at this stage, either in cost or in design, it may be necessary to reinvestigate the cost-effectiveness of the solution.

First, the relative importance of each criterion is assigned by the design team in consultation with RDA. Next, the alternatives are rated according to their anticipated performance in the criterion areas. Then, an alternative’s rating in an area is multiplied by the assigned weight of that factor to achieve a “score”. Finally, all of the scores for an alternative are summed, and the alternative with the highest score is the preferred solution.

![Figure 6.1: Illustrative Method of Selecting Rehabilitation Alternatives (from Ref 3)](image_url)
14.7 DESIGN OF RIGID PAVEMENTS

A general methodology of rigid pavement design is presented in Figure 2.

14.7.1 DESIGN TRAFFIC LOADING

Refer to Chapter 3 and to subsection 6-2 of this volume for the computation of the design Equivalency Axle Load of the road.

14.7.2 THICKNESS DESIGN

- Capping and Subbase

The capping layer is required only if CBR of the subgrade is 15% or less. The required thickness of a capping layer for a CBR value less than 15% can be obtained from Figure 3.

The subbase layer is required when the subgrade material doesn’t comply with the requirement for a subbase (CBR is less than 30%) or to facilitate the obtaining of the surface levels with the tolerances required. Generally, the thickness of the subbase provided will be a constant 15 cm and can be cement stabilized.

For subgrade CBR values less than 2%, the roadbed material needs to be treated either by replacement or in-situ stabilization. These methods of soil improvement are described in section 7 of the flexible pavement design manual.

A separation membrane (such as a polythene sheet) is required between subbase and concrete slab, mainly in order to reduce the friction between the slab and the subbase in JUCP and JRCP pavements, and thus inhibits the formation of mid-bay cracks. It also reduces the loss of water from the fresh concrete. For CRCP pavements, a bituminous spray should be used on the subbase, instead of polythene, because a degree of restraint is required.

- Concrete Slab Thickness and Reinforcement

Based on the design traffic volume determined as per Chapter 2 and project-specific characteristics, the thickness of pavement is determined.

The following represents procedures for determining the thickness and reinforcement for each of the pavement types.

Jointed Unreinforced Concrete Pavement (JUCP)

For a given traffic volume in terms of ESAs, the thickness of JUCP concrete slab can be determined using Figure 4.

Figure 4 assumes the presence of an effective lateral support to the edge of the most heavily-trafficked lane (i.e., the right lane), such as a shoulder with a pavement structure able to carry occasional loads. In the absence of such a shoulder adjacent to the most
heavily trafficked lane, an additional slab thickness is required, and this additional thickness can be determined using Figure 6.

JUCP pavements have no reinforcements. However, the longitudinal and transverse joints are provided with reinforcements. The joint details are discussed in Section 5

**Jointed Reinforced Concrete Pavement (JRPC)**

For a given traffic volume in terms of ESAs, the thickness of a JRPC concrete slab can be determined using Figure 4. The figure can also be used to determine the longitudinal reinforcement in terms of mm²/m for a design thickness of concrete slab. Thus, several alternate combinations of thickness of concrete slab and amount of reinforcement can be compared.

In the absence of an effective lateral support provided by the shoulder adjacent to the most heavily trafficked lane, an additional slab thickness is required and can be determined using Figure 6.

In addition to the longitudinal reinforcement, JRPC pavements shall be provided with transverse reinforcement, if required, depending on site conditions. In that case, reinforcement shall be provided at 600 mm spacing and consist of 12 mm diameter steel bars.

**Continuously Reinforced Concrete Pavement (CRCP)**

CRCP pavement can withstand severe stresses induced by differential movements. For a given traffic volume, in terms of ESAs, the thickness of CRCP concrete slab can be obtained from Figure 5.

Longitudinal reinforcement in CRCP pavements shall be 0.6% of the concrete slab cross-sectional area, consisting of 16 mm diameter deformed steel bars. If required, transverse reinforcement shall be provided at 600 mm spacings, consisting of 12 mm diameter deformed steel bars, to control the width of any longitudinal cracks which may form. Transverse reinforcement is normally required only for ease of construction. It may be omitted except where there is a risk of differential settlements.

Similarly to JUCP and JRPC pavements, in the absence of effective shoulder support adjacent to the most heavily trafficked lane, the additional slab thickness required can be determined using Figure 6.

As is evident from Figure 4 and 5, the minimum thickness of concrete pavement for JUCP and JRPC pavement is 150 mm and that for CRCP pavement is 200 mm. Hence, the designer should carefully assess the necessity and requirements for such pavements, depending on the design traffic volume, and shall include flexible pavement as an alternate.

A design example of rigid pavement design is presented in Appendix C.

14.7.3 DESIGN FOR MOVEMENT

Joints shall be designed according to the general considerations of sub section 2.2 and using Drawings B2 to B8.
The general layout of joints shall account for construction consideration and the following limitations concerning joint spacing and slabs dimensions:

- **Transverse Joint Spacing**

  Maximum transverse joint spacing for JUCP pavements is 4 m for slab thickness up to 230 mm and is 5 m for slab thickness over 230 mm.

  Expansion joints should replace every third contraction joint, i.e. at a spacing of 12m or 15m.

  For JRCP, contraction joints are generally at a standard distance of 25m, unless there is 500 mm²/m of reinforcement. Then refer to Figure 4. For expansion joints, replace every third contraction joint with an expansion joint. For example, a pavement with contraction joint spacing of 25m has an expansion joint spacing of 75m.

- **Longitudinal joint spacing**

  The longitudinal joint spacing shall not be greater than 4.2 m for pavement slabs without transverse reinforcement and 6.0 m for pavement slabs with transverse reinforcement. When required, longitudinal joints shall be placed at the edge of traffic lanes.

- **Slabs dimensions**

  Warping joints shall be added to the general layout in special cases, as described in section 2.2. In general, the length/width ratio of the slabs shall be of 2 or less.

- **Layout at junctions**

  For intersection of joints at junctions and crossings, refer to the drawing below:

  ![Joints layout at junctions and crossings](image)
Check for Requirement of Capping Layer

No

Determine Capping Layer Thickness from Figure 3.2

Check for Requirement of Subbase Layer

Check for Site Constraints, if any. Determine pavement type.

Determine Concrete Slab Thickness and Reinforcement from Figure 3.3.

Check for requirement of transverse reinforcement (cf. § 3.2).

Determine Concrete Slab Thickness from Figure 3.4.

Determine slab thickness from Figure 3.3.

Determine additional slab thickness from Figure 3.5.

Determine joint type, joint spacing and joint reinforcement.

Yes

Determine Design CBR Design ESAs

CRCP

Determine longitudinal reinforcement (cf. § 3.2)

JRCP

Determine longitudinal reinforcement (cf. § 3.2)

JUCP

Determine joint type, joint spacing and joint reinforcement.

Figure 2 Design Methodology – Flow Diagram
Figure 3  Capping Layer and Subbase Thickness design
(Adapted from Ref.5)
Figure 4  Design Thickness and joint spacing for JUCP and JRCP  
(Adapted from Ref. 5)
Figure 5  Design Thickness for CRCP Pavements  
(Adapted from Ref. 5)

Figure 3.5  Additional Concrete Slab Thickness for Rigid Pavements  
Without Lateral Support (from Ref. 5)
## APPENDIX A  APPLICABLE STANDARDS

<table>
<thead>
<tr>
<th>Designation</th>
<th>Name of Test</th>
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<tr>
<td><strong>Tests on Soils</strong></td>
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<tr>
<td>ASTM D698</td>
<td>Moisture-Density Relations of Soils/Soil-Aggregate Mix (Light)</td>
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<tr>
<td>ASTM D1557</td>
<td>Moisture-Density Relations Using Modified Effort (Heavy)</td>
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<td>AASHTO T180</td>
<td>Compaction Test</td>
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<td>CBR Test for Laboratory-Compacted Soils</td>
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<td>ASTM C136 or AASHTO T27</td>
<td>Particle Size Distribution</td>
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<tr>
<td>ASTM D424 or AASHTO T90</td>
<td>Plastic Limit and Plasticity Index</td>
</tr>
<tr>
<td>ASTM D2487 or AASHTO M145</td>
<td>Classification of Soils for Engineering Purposes</td>
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<tr>
<td><strong>Tests on Gravels</strong></td>
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<td>Moisture-Density Relations Using Modified Effort</td>
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<td>AASHTO T180</td>
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<td>Liquid Limit</td>
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<td>ASTM D424 or AASHTO T90</td>
<td>Plastic Limit and Plasticity Index</td>
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<td>Magnesium Soundness Test (SST)</td>
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<td>ASTM C128</td>
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<td>ASTM D2872</td>
<td>Rolling Thin Film Oven Test (TFOT)</td>
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<tr>
<td>ASTM D1559 or</td>
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<td>AASHTO T245</td>
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<td>ASTM D4402</td>
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<td>ASTM D5</td>
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<td>ASTM D3910</td>
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<td>TRL ORN 31</td>
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<td>TRL ORN 3</td>
<td>DCP Test</td>
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<td>TRL ORN 3</td>
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<td>TRL ORN 3</td>
<td>Probe Penetration Test</td>
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<tr>
<td>ASTM D3497</td>
<td>Dynamic Modulus of Bituminous Material</td>
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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

<table>
<thead>
<tr>
<th>P1</th>
<th>Correction factor SF</th>
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<tr>
<td>0</td>
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<tr>
<td>10</td>
<td>0.3</td>
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<tr>
<td>15</td>
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<td>1.6</td>
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<td>2.0</td>
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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-1, 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°C should be used as the test temperature.

SUGGESTED NEW PROCEDURE

For Somaliland 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°C
APPENDIX G  RIGID PAVEMENT DESIGN - DESIGN EXAMPLE

Design of Rigid Pavement for the following data:

- Design period: 20 years
- Cumulative number of ESAs over the design period (given): $40 \times 10^6$
- CBR of roadbed material: 8%
- Width of paved carriageway: 7.3 m

Step 1 Check for Capping Layer

Since the CBR value is less than 15%, a capping layer is required.

Referring to Figure 3, the required thickness of the capping layer for a subgrade CBR of 8% is 200 mm.

Step 2 Subbase

The thickness of the required subbase layer (Cf. § 3.1) is 150 mm.

Step 3 Pavement Slab

From Figure 4, for a design traffic volume of $40 \times 10^6$ ESAs and a longitudinal reinforcement of $500 \text{ mm}^2/\text{m}$, the thickness of pavement slab required is 200 mm.

Step 4 Joints

From Figure 4, the maximum transverse joint spacing is 25 m. Since the thickness of concrete slab is 200 mm (<239 mm), the dowel bars shall be 20 mm in diameter at 300 mm spacing, 400 mm long. Depending on site conditions, expansion joints may be provided, if required.

The conditions are assumed to be such that no transverse reinforcement is deemed necessary. As a result, the maximum longitudinal joint spacing is 4.2 m and we place one longitudinal joint in the middle of the carriageway, between the two lanes (3.65 m each). The tie bars for this slab (<239 mm in thickness) shall be 12 mm in diameter at 600 mm spacing, 1000 mm long.

Step 5 Final Design

The pavement cross-section is shown on the next page.

- Longitudinal reinforcement: $500 \text{ mm}^2/\text{m}$
- Transverse joint spacing: 25 m
- Longitudinal joint spacing: 3.65 m
- Dowels for transverse joints: 20 mm diameter @ 3000 mm c/c, 400 mm long
- Tie rods for longitudinal joints: 12 mm diameter @ 600 mm c/c, 1000 mm long
Rigid Pavement Cross-Section
APPENDIX H

VIZIR METHOD
FOR QUALITY EVALUATION OF PAVED ROADS

CLASSIFICATION AND QUANTIFICATION OF DAMAGES

GENERAL

The first part of VIZIR is a method of classification and quantification of damage that forms part of general road maintenance management studies or special route maintenance studies. It is intended to provide a picture of the surface condition of a road at a given time and to identify zones of equal quality (in terms of three damage levels).

These zones of equal quality and these three damage levels are used to determine the nature and type of work required; in some cases, the very identification of the damage determines the solution, while in others it is only one factor in a more complex diagnosis involving other criteria.

The damage classified in the VIZIR method is relevant primarily to flexible pavements with bituminous surfacings. This damage is divided into two categories:

TYPE A DAMAGE

This characterizes the structural condition of a pavement, affecting either all of its courses and the ground or the surfacing only. This damage is caused by a structural deficiency of the pavement, and its identification is used in the search for a solution in conjunction with other criteria, in particular the bearing capacity as characterized by the static deflection.

Type A damage includes four types:

- deformation
- rutting
- (fatigue) cracking
- crazing

TYPE B DAMAGE

This damage leads to repairs that are generally unrelated to the pavement’s structural capacity. It may be caused by defective placement, by deficient product quality, or by some special local condition, possibly aggravated by traffic.

Type B damage includes:

- cracking other than fatigue cracking, i.e. longitudinal joint cracks, transverse thermal shrinkage cracks, and longitudinal and transverse clay shrinkage (desiccation) cracks,
- potholes,
- raveling and, more generally, all surfacing defects such as fretting, bleeding, etc.
SURVEY AND GRADING OF DAMAGE

The damage is surveyed by an operator who travels the length of the route and records, for any damage:

- its type,
- its severity,
- its extent, i.e., the length of road affected or, as appropriate, the area.

VIZIR provides a damage topology and, for each type of damage, three levels of severity.

The survey can be done manually, while travelling the road on foot or by car. The operator in this case enters his observations (identification of damage and estimate of its severity) on a route diagram, a document representing the route as a straight line, the scale and precision of which are appropriate to the type of study.

The survey may also be done using the LCPC’s DESYROUTE equipment. Carried in a car, it makes it easy to record the type, severity, and location of any damage (position on curved x-axis based on information from the distance indicator). Its data entry and retrieval software allows all sorts of calculations, such as a mean severity index and an extent as a percentage of a given length, or an overall quality index based on a combination of several types of damage. It can also read out all of these data, either in the form of a route diagram identical to the one produced by hand or in the form of files that can be incorporated in a road data bank (for instance the VISAGE road data bank).

To help the operator in his work, VIZIR proposes a catalogue of damage and a method of graphic representation that are summed up in Table 1 (Type A damage) and Table 2 (Type B damage).

In the route diagram, damage is represented by a rectangle of which the background (white, grayish, or black) indicates the level of severity, while the two sides represent the co-ordinates of the beginning and end of the damaged zone, or in other words, its extent.

The severity values given in Tables 1 and 2 are average values suitable for many road networks. However, they can be altered to fit the assigned maintenance objectives and the expected level of severity.
Each type of damage and each associated level of severity, as indicated in Tables 1 and 2, is described in detail in the next sub-section and shown in a photograph attached to the present appendix.
DESCRIPTION OF TYPE A DAMAGES

This concerns the following types of damage:

1. Deformation, rutting
2. Fatigue cracking and crazing
3. Patching and repairs

DEFORMATION AND RUTTING

The deformations specific to flexible pavements almost always lead to rutting or subsidence. Their degree of severity is determined by the depth h measured on a straightedge 1.5 m long placed crosswise on the pavement (see following figure).

![Figure H.1 Measurements of Deformation](image)

Rutting appears in the wheel tracks, on the back of the pavement about 50 to 80 cm from the edge. It is caused by settlement of the materials under heavy, channelized traffic, possibly but not necessarily aggravated by a deficiency of bearing capacity of the soil. There may also be rutting through creep of the bituminous courses only, but this is found most often in semi-rigid pavement or in pavements with a bituminous roadbase.

The following values are given for guidance:

<table>
<thead>
<tr>
<th>Severity</th>
<th>Description</th>
<th>Image</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>h &lt; 2 cm</td>
<td>Photo 1</td>
</tr>
<tr>
<td>2</td>
<td>2 cm &lt; h &lt; 4 cm</td>
<td>Photo 2</td>
</tr>
<tr>
<td>3</td>
<td>h &gt; 4 cm</td>
<td>Photo 3</td>
</tr>
</tbody>
</table>

Subsidence affects the entire edge of the pavement. It is a result of deficiencies of bearing capacity or of stability, possibly caused by materials of poor quality or excessively high water contents. Subsidence often occurs in bends and in zones of very high stress:

Localized subsidence of severity 1
Ridge of severity 3
Subsidence of severity 3

Longitudinal deformations generally show up in the evenness measurement (sag of a few cm, wavelength from 1 to 20 cm) and must not be counted twice. However, longitudinal
deformation in the form of an isolated ridge may properly be recorded with transverse deformations.

**Fatigue Cracking**

This category does not include cracks resulting from faulty construction, such as the longitudinal joint between two mix spreading bands, or cracks caused by some particular behavior of the material, such as longitudinal or transverse thermal shrinkage cracks or clay shrinkage cracks.

Longitudinal fatigue cracks, on the other hand, are recorded. Most often initially single and isolated, they evolve toward continuous cracking, sometimes branching, before multiplying in the wheel tracks to the point of becoming very closely spaced.

**Severity 1**  Single or clearly separated longitudinal cracks  
**Photos 7 and 10**

**Severity 2**  Continuous cracks, branched or clearly open  
**Photos 8 and 11**

**Severity 3**  Extensively branched cracks foreshadowing crazing or wide open cracks  
**Photos 9 and 12**

**Patching and Repairs**

In some damage survey methods, repairs may not be counted. This is true of the *OECD Manual– Road monitoring for question of maintenance – DC1 – 1990*. The LCPC has always held that repairs, like damage, are an integral part of the visual examination. The 1977 guide to the examination of flexible pavements, in fact, says, “a recent repair conceals a problem; frequent repairs call attention to it”. Repairs are intended to palliate the deficiencies of a pavement, temporarily or permanently; their number, extent, and frequency in time are elements of the diagnosis. In the VIZIR method, repairs must be rated in the course of the visual examination, because some of them are used in determining the index of pavement appearance. Two cases are distinguished:

- The repair has definitively eliminated the defect, in which case it will appear on the survey document (route diagram) but not be counted when calculating the surface quality index;
- The repair has more or less adequately eliminated the defect, but not its cause, and will doubtless reappear here or there or nearby; the repair is therefore reflected as an aggravating factor in calculating the visual index.

The first category includes repairs of Type B defects or partial or complete rebuilding of the pavement, in good condition. VIZIR rates them as being of severity 1. Rebuilding may be of the surfacing only or of the whole pavement:

- Rebuilding of wearing course at longitudinal joint (Type B damage)  
  **Photo 19**

- Rebuilding of pavement (Type A damage)  
  **Photos 20 and 21**
Patching of the wearing course (Type B damage)  

Structural defects of pavement after repair (Type A)  

The second category includes localised pavement repairs of Type A defects, classed as severity 2 if in good condition and severity 3 otherwise. Only severities 2 and 3 may, according to their extent, increase the visual index. Photos 21 and 24 illustrate such cases.

Table H.3  Extent of Patching and Repair vs. Severity

<table>
<thead>
<tr>
<th>Severity</th>
<th>Extent</th>
<th>0 to 10%</th>
<th>10 to 50%</th>
<th>&gt; 50%</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>+1</td>
</tr>
<tr>
<td>3</td>
<td>0</td>
<td>+1</td>
<td>+1</td>
<td></td>
</tr>
</tbody>
</table>

Table for calculating the correction of surface damage I, according to the severity and extent of repairs.

DESCRIPTION OF TYPE B DAMAGES

The common Type B damage in flexible pavements includes:

- Cracking of longitudinal joint;
- Potholes
- Raveling and, more generally, any surface defect such as fretting, bleeding, etc.

There are others that depend on extreme climatic conditions or particular materials, such as:

- Thermal shrinkage of bituminous materials;
- Clay shrinkage of pavement courses.

These latter damages do not occur on the Ethiopian network. VIZIR is intended for flexible pavements, and so does not cover the thermal shrinkage cracking of hydraulic materials. If they had to be included, they would be in category B.

CRACKING OF LONGITUDINAL JOINT

This is a failure of bonding between two adjacent bands of coated materials. This type of crack is initially single and straight, and can be repaired by treatment specific to such cracking (severity 1). Traffic causes the crack to branch or double and lose material at the edges (severity 2, 3). Repairs are more costly: complete rebuilding of the damaged zone or placement of a new wearing course.

<table>
<thead>
<tr>
<th>Severity</th>
<th>Description</th>
<th>Photo</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Single hair line crack</td>
<td>16</td>
</tr>
<tr>
<td>2</td>
<td>Branching crack</td>
<td>17</td>
</tr>
</tbody>
</table>

Photo 22
Photos 23 and 24
Severity 3 Wide, branching crack with loss of materials  

POTHOLEs

Potholes most often result from wear or destruction of the wearing course, sometimes from the presence of foreign bodies in the surfacing.

They are small when they first appear. In the absence of maintenance, they grow and reproduce in rows, often with a pitch equal to the circumference of a truck type.

Depending on the degree of severity of these potholes, it may be necessary to rebuild part of the wearing course or, if they have become too deep, of the wearing course and roadbase. In some cases, it may even be necessary to rebuild the whole pavement. If there are too many potholes, it may be necessary to rebuild a considerable length of pavement.

The three levels of severity depend on the nature of the work:

Severity 1 Small number of potholes that can be plugged by ordinary patching  

Severity 2 Large number of small potholes in surface  

Severity 3 Potholes or crazing formation – pavement must be rebuilt or overlaid

MOVEMENT OF MATERIAL: RAVELLING, fretting, bleeding, etc.

Movements of materials include ravelling and bleeding: some ravelling is caused by excessive wear of the wearing course and may lead to the formation of potholes. It is normally counted as cracking or crazing.

Ravelling of severity 1 on crazing of severity 2, counted as Type A damage

Other ravelling is caused by defective construction. The following damages are distinguished:

- **Ravelling:** more or less large losses of gravel from surface dressings (deficiency of bonding, hence of the quality and/or quantity of bitumen);
- **Scabbing:** loss of all or part of a thin (<3 cm) wearing course of coated materials and separation from its substrate; this is often preceded by major cracking of the wearing course;
- **Fretting:** deficient implementation due to bad equipment operation: poor distribution of bitumen by the sprayer, hence surface unequally covered by the bitumen or bad gravel distribution on the pavement.
- **Bleeding:** upward movement of excess bitumen in hot weather

Severity 1 Discontinuous ravelling of plucking type  

Severity 2 Continuous ravelling
There are other forms of damage that are specific to a climate, a country, or a given traffic pattern and that may require rebuilding of the shoulders as part of maintenance:

- **Lacy edge:** this damage occurs in pavements in which the roadbase and shoulders are of the same type, and is caused by the frequent stopping of vehicles on the shoulders. The extent of the damage is more important than its severity.

  - **Severity 1**  Onset of lacy edges  
  - **Severity 2**  Lace cutting more than 0.50 m into the pavement  
  - **Severity 3**  Extreme erosion approaching destruction of the pavement

- **Low shoulders:** this damage is caused by maintenance of the shoulders, which gradually become lower than the pavement surface.

  - **Severity 1**  From 1 to 5 cm lower  
  - **Severity 2**  From 5 to 10 cm lower  
  - **Severity 3**  More than 10 cm lower

- **Ditch and shoulder erosion:** the erosion may take a number of forms (rainwash of ditch; rainwash and destruction of ditch; destruction gravely endangering a part of the pavement). The severity code follows this progression:

  - **Severity 1**  Erosion of ditch – repairs limited to ditch  
  - **Severity 2**  Erosion and rainwash of shoulder  
  - **Severity 3**  Threat to or destruction of a part of the pavement
**RUTTING**

Severity 1
- The sag under a 1.5 m straightedge is less than 2 cm.

Severity 2
- The sag is less than 4 cm.

Severity 3
- The sag is greater than 5 cm.

**DEFORMATION**

Severity 1
- Localized subsidence revealed by puddle

Severity 3
- Subsidence over a long stretch of pavement, with depths exceeding 8 cm.

**CRACKING**

Photo 3
- The sag is greater than 5 cm.

Photo 5
- Ridge in centre of pavement; more than 10 cm thick but localized
Severity 1

Photo 7
Single hairline longitudinal crack

Severity 1

Photo 10
Longitudinal cracks

Severity 2

Photo 8
Branching longitudinal crack

Severity 2

Photo 11
Clearly open longitudinal crack

Severity 3

Photo 9
Highly branching longitudinal crack foreshadowing crazing

Severity 3

Photo 12
Branching and very open longitudinal crack
CRACKING AND CRAZING

Severity 1
Photo 13
Fine crazing with large map cracking

Severity 2
Photo 14
Crazing with medium map cracking, cracks clearly open.

Severity 3
Photo 15
Generalized crazing with fish net cracking; losses of materials with

CRACKING OF LONGITUDINAL JOINT

Severity 1
Photo 16
Single fine centreline crack, straight

Severity 2
Photo 17
Branching centreline crack, no stripping.

Severity 3
Photo 18
Wide branching centreline crack stripping
PATCHING

Severity 1

Photo 19
Repair of wearing course:
(Type B defect) A

Severity 3

Photos 20-21
Structural defects of pavement after repairs.

Severity 3

Photo 24
Potholes forming in pavement must be rebuilt or overlaid.

POTHOLES

Severity 1

patching Photo 22
few small potholes

Severity 2

Photo 23
Potholes small and shallow, but numerous.

Severity 3

crazing;

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RAVELLING, FRETTING, BLEEDING

Severity 1

Isolated stripping on crazing type.

Photo 25

Isolated stripping of scabbing

Photo 28

Severity 1

Discontinuous stripping of plucking type

Photo 26

Very marked generalized fretting

Severity 2

Continuous stripping

Photo 27

Very marked continuous bleeding in wheel tracks.

Photo 30

Severity 2

Very marked generalized fretting

Photo 29
LACY EDGE

Severity 1

Photo 31
Lacy edge forming; damage not clearly marked

Severity 2

Photo 34
Shoulder substantially lower than pavement (up to 10 cm).

Severity 2

Photo 32
Continuous marked lacy edge

Severity 3

Photo 33
Disappearance of pavement.

Severity 3

Photo 36
Destruction by undermining of shoulder.

LOW SHOULDER

Severity 1

Photo 35
Erosion of ditch.
SURFACE DAMAGE

Rating 2

- Hairline cracking in wheel tracks (severity 1), not very extensive. If = 1.
- Pavement not deformed; Id = 0.

Rating 3

- Pavement not cracked; If = 0
- Highly localized rutting of pavement, rut depth < 4 cm (Severity 2); Id = 2.

Rating 2

- Hair line cracking in wheel tracks and in vehicle centreline (Severity 1) rather extensive; If = 2
- Pavement not deformed; Id = 0.

Rating 4

- Continuous branching cracking (severity 2); If = 4
- Pavement not deformed; Id = 0.

Rating 3

Pavement crazed with highly localized deformation.
- Closely-spaced crazing (severity 2); If = 2
- Deformation < 4 cm (severity 2); Id = 2

Rating 4

- Pavement not cracked; If = 0.
- Pavement slightly deformed (severity 1), continuously; Id = 3
Appendix

Rating 5

- Closely-spaced crazing with losses of materials (severity 3), continuous, If = 5.
- Very slight localized deformation of pavement; Id = 1.

Rating 6

- Pavement not cracked, continuously deformed longitudinally (severity 3); If = 0.
- Pavement continuously repaired.

Rating 7

- Pavement badly cracked and locally deformed, hair line continuously rather large and formed Intermediate rating 4.
- Systematic repair of with edges of pavement. (severity 2); If = 4.
- Final rating 5.

Rating 5

- Pavement badly cracked and locally deformed, hair line continuously rather large and formed Intermediate rating 4.
- Systematic repair of with edges of pavement. (severity 2); If = 4.

Rating 6

- Pavement continuously crazed and deformed.
- Rather closely spaced crazing with some losses of materials (severity 2), If = 4.
- Slight rutting (severity 1); Id = 3

Rating 7

- Pavement badly crazed (severity 3) and deeply rutted (severity 3) continuously; If = 5
- Deep rutting (severity); Id=5.
Deformation due to loss of material

Severity 1

Severity 2

Severity 3

Deformation due to subsidence
Deformation due to rutting

Severity 1

Severity 2

Severity 3

Potholes
Corrugation

Severity 1

Severity 2

Severity 3

Gullies
Use of Damage Quantifiers to Determine Maintenance Needs

**GENERAL**

In the previous chapter, two types of damage have been distinguished (A and B). For each type of damage, a scale of classification according to three levels of severity has been provided. Each type of damage is accordingly quantified by two estimators:

- its extent (length of road affected);
- its severity.

VIZIR quantifies damage to estimate the quality of a pavement, investigated either in connection with a road maintenance management study (network) or with a view to particular work (route). In both cases, even if the levels of precision are different, rehabilitation approaches must be identified.

In the case of type B damage, the maintenance approach follows from the identification of the damage by itself, and no other parameters are needed for the diagnosis. For example, centerline cracking requires bridging of the cracks, lacy edges require rebuilding of the edges and shoulders, and so on.

This is not true of type A damage; the approach will depend on other factors, and the diagnosis will be based on damage, bearing capacity, traffic, etc. It is therefore necessary to establish an overall rating of visual condition, similar to the ratings or classes used for the other parameters. Type A damage leads to major work such as rebuilding or overlaying of the surfacing. This work remedies type B damage in passing: bridging a centerline crack to prevent the infiltration of water is pointless if a wearing course is going to be applied over the pavement. Generally, type B damage influences the type of work done only if there is no type A damage (whence the choice of the order A and B) and, conversely, the global visual index used to qualify the pavement counts only type A damage.

**DAMAGE INDEX IS**

The global visual index IS is calculated for a specified length of road from three damage groups:

- cracking and crazing;
- deformation and rutting;
- repairs.

A cracking index IC, depending on the severity and extent of cracking or crazing of the length of road in question, is calculated first. When both cracking and crazing are present, the larger of both values is used.

A deformation index ID, depending on the severity and extent of deformation and rutting is then calculated in a similar manner.
If and I_d are combined in a first pavement quality index. It may, as appropriate, be corrected to reflect the severity and extent of certain repairs. It has already been pointed out that some repairs conceal pavement deficiencies, and so are treated as aggravating factors when estimating surface quality.

This correction yields a global damage index I_s that qualifies the pavement over the length chosen for the calculation. I_s ranges from 1 to 7. Ratings 1 and 2 reflect good surface conditions that need no work (or at least on which work may safely be postponed). Ratings 3 and 4 represent a rather intermediate surface condition, bad enough to trigger maintenance work in the absence of any other consideration. Ratings 5, 6 and 7 represent very poor surface conditions requiring major maintenance or overlay work.

The baseline length on which I_s is calculated may depend on the type of study, the database, other parameters included in the diagnosis, and the operator.

For studies of road maintenance management systems, which are global studies, the route diagram is plotted to a scale of about 2 cm per km; the database itself is established with a step on the order of 500 m. I_s may therefore be calculated for 500 m lengths.

For route maintenance planning, the survey is plotted to a scale of about 5 cm per km, and a 200 m step may be used for the calculation.

Finally, in the case of a test section, the route diagram is plotted to a scale of about 20 cm per km; a high degree of precision is attained and I_s may be calculated for 50 m lengths.

When the damage survey is done using DESYROUTE, the software of which includes VIZIR, simply indicate, during data processing, the measurement length on which the calculation should be based and the scale on which the route diagram should be printed out.
Figure 2  Principles of Determination of Surface Condition Rating

- Extent and severity of cracking: cracking index
- Extent and severity of deformation: deformation index

Surface condition rating

Possible correction for repairs

Damage index $I_s$
### Visual Examination

#### Cracking Index

<table>
<thead>
<tr>
<th>Extent</th>
<th>Severity</th>
<th>0 to 10%</th>
<th>10 to 50%</th>
<th>&gt;50%</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>1</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td></td>
</tr>
</tbody>
</table>

#### Deformation Index

<table>
<thead>
<tr>
<th>Extent</th>
<th>Severity</th>
<th>0 to 10%</th>
<th>10 to 50%</th>
<th>&gt;50%</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>1</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td></td>
</tr>
</tbody>
</table>

#### Correction for repairs

<table>
<thead>
<tr>
<th>Extent</th>
<th>Severity</th>
<th>0 to 10%</th>
<th>10 to 50%</th>
<th>&gt;50%</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>0</td>
<td>+1</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>0</td>
<td>+1</td>
<td>+1</td>
<td></td>
</tr>
</tbody>
</table>

(1) Separate calculation for longitudinal cracking and crazing. The larger of the two indices is used.

#### First damage index

<table>
<thead>
<tr>
<th>Id</th>
<th>0</th>
<th>1-2</th>
<th>3</th>
<th>4-5</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>1-2</td>
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<td>3</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>3</td>
<td>4</td>
<td>5</td>
<td>5</td>
<td>6</td>
</tr>
<tr>
<td>4-5</td>
<td>5</td>
<td>6</td>
<td>7</td>
<td>7</td>
</tr>
</tbody>
</table>

**Figure 3.1 (from Ref. 12)**
DEVELOPMENT OF SOLUTIONS

GENERAL

The development of maintenance or overlay solutions from a diagnosis combining a visual examination with other parameters such as bearing capacity and traffic is not independent of the context of the study. This section deals with the search for solutions in the context of road maintenance management systems.

The methodology of the studies performed by the LCPC may be represented schematically as follows:

a. Construction of a database, field survey of data, determination of network quality at time $t_0$

b. Analysis of data, determination of “technical solution”, i.e. what should be done to the road network on the basis of its quality image at time $t_0$ to restore a certain level of service, in the absence of any budgetary constraints.

c. Search for an “optimised solution” reflecting both technical requirements and budgetary constraints, including staging of the work over several years, and time projection of the image of the network at time $t_0$ and of the technical solution. For this, laws describing change against time must be available. The World Bank’s HDM III model proposes such laws for developing countries.

The technical solution is determined in two stages.

- First, “visual condition” and “bearing capacity” are combined; the bearing capacity is indicated by the deflection value and the visual condition is represented by index $I_s$ as determined in the previous section. The combination generates a pavement quality rating on a scale from 1 to 9 and a conclusion as to what needs to be done: nothing, maintenance or an overlay.

- Then, in a second stage, the pavement quality rating and the level of traffic are combined and the work to be done is decided for each pair of values.

PAVEMENT QUALITY RATING $Q_i$

The pavement quality rating $Q_i$ is estimated by combining the value of the Index $I_s$ qualifying the pavement surface and the deflection value qualifying the combined bearing capacity of the pavement and foundation soil.

- Damage Index $I_s$

The damage index is divided into three ranges:

**Rating 1 or 2** Little or no cracking or deformation; good surface condition requiring no (or only just requiring) immediate maintenance.
**Rating 3 or 4**  
Cracks with little or no deformation, or deformation without cracking; intermediate surface condition, bad enough to trigger maintenance work in the absence of any other consideration.

**Rating 5, 6 or 7**  
Extensive cracking and deformation; poor surface condition requiring major maintenance or overlay work.

- **Deflection**

Deflection is also divided into three ranges, by thresholds d1 and d2:

- **d1**  
  value above which pavement performance is generally good

- **d2**  
  value above which pavement performance is poor

- **d1-d2**  
  range of indecision.

The choice of thresholds d1 and d2 depends on many factors, such as climate, type and thickness of pavement, soils, axle loads, and so on. These values are generally based on a particular country’s experience. They may, where possible, be based on the examination of control sections during the general study of the network. If values based on experience are not available in a country, this calibration may be done during the study. As far as Ethiopia is concerned, the issue of the determination of d1 and d2 is discussed in Part III of the present manual.

Table 4 gives the pavement quality rating $Q_i$ as a function of the Index $I_s$ and the deflection value. Three main ranges have to be considered:

- **Q1, Q2, and Q3**  
  these ratings mean that there is nothing that needs to be done, or at most maintenance work depending on the traffic carried by the road. When sealing is required, the cracking index is used to determine the data and type of work.

- **Q7, Q8, and Q9**  
  these ratings mean that the pavement requires an overlay, the thickness of which is determined by the traffic.

- **Q4, Q5, and Q6**  
  this is a zone of indetermination for which the index $I_s$ and the deflection value seem inconsistent. These cases should be further examined and eventually reclassified. The procedure is as follows:

- **Q4**  
  pavement with marked damage in spite of a good bearing capacity. The validity of the deflection measurement should be checked together with the nature of the damages (in particular rutting layers of coated materials, unrelated to the deflection measurement). Depending upon the answer, Q4 will be reclassified as Q2 (priority to deflection) or Q7 (priority to damage).

- **Q5**  
  same analysis as above; allowance will be made for the position of the deflection with respect to the limits and to the traffic; depending upon the answer, may be reclassified as Q3, Q7, or Q8.
Q6: pavement having a large deflection value without apparent damage; to validate or unvalidate the surface condition, check the age of the pavement or the date of the most recent work, together with the traffic level. Depending upon the answer, may be reclassified as Q3 or Q8.

* or other rehabilitation method

Figure 3.2 Pavement Quality Rating (Qi) and Required Road Works
(adapted from Ref. 5)
APPENDIX I

FIELDWORK

PAVEMENT CONDITION SURVEYS

Condition surveys are essentially required to assess a pavement’s physical distress and form the basis of a diagnosis regarding the maintenance or rehabilitation needs. Together with drainage, destructive and non-destructive testing, they are mandatory before rehabilitation design.

Flexible Pavements

One of the stated objectives of the PMS within RDA is to provide a pavement condition evaluation system. A condition survey conducted for a specific project, in order to be compatible with the PMS of the network, must follow the same method. Also, by using the same method, the results of the specific condition survey can be forwarded to the PMS Branch and incorporated into the PMS Road Data Bank.

Following is a summarized description of the method of classification and quantification of visible damage, adapted from the “VIZIR method”, as presented in the Manual of Procedures for PMS (Ref 12) and copied in extension for reference in Appendix A.

General

The method of classification and quantification of damage is intended to provide a picture of the road surface condition at the time of inspection and to identify zones of equal quality (in terms of three damage levels). The damage is divided into two categories:

- Type A damage, which characterizes the structural condition of the pavement and includes four types as follows:
  - deformation
  - rutting
  - (fatigue) cracking
  - crazing
- Type B damage, generally unrelated to the pavement structural capacity. This type of damage may be caused either by defective placement, or by deficient materials quality, or by some special local condition, aggravated by traffic. Type B damage includes:
  - cracking other than fatigue cracking, i.e. longitudinal joints and transverse thermal shrinkage cracks,
  - potholes,
  - ravelling and, more generally, all surfacing defects such as fretting, bleeding, etc.

Survey and Grading of Damage

The survey is intended to record, for any damage:

- its type
- its severity
- its extent, i.e. the length of road affected, or, as appropriate, the area
It is to be noted that the method was designed for use with automatic data recording equipment, which facilitates the recording of the distresses and the calculation of quality ratings. Nevertheless, the survey can also be done manually. Two options are therefore offered to the team in charge of the pavement condition survey:

- Hiring the services of RDA’s PMS Branch personnel and use of its equipment;
- Performing with its own forces the survey manually, while travelling the road on foot or by car.

The severity of each type of damage is described in detail, in tabulated form as well as by illustrations (photographs) in Appendix A. Following are some examples.

**Description of Type A Damage**

Rutting appears in wheel tracks about 50 to 80 cm from the pavement edge. It may be due to settlement of the materials under heavily channelized traffic, or occasionally due to creep within the bituminous layers only.

As an example, rutting is characterized by the following levels of severity:

<table>
<thead>
<tr>
<th>Severity</th>
<th>h</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>h &lt; 2 cm</td>
</tr>
<tr>
<td>2</td>
<td>2 cm &lt; h</td>
</tr>
<tr>
<td>3</td>
<td>4 cm &lt; h</td>
</tr>
</tbody>
</table>

Where h is the depth of rutting measured by a straight edge. As indicated earlier, complete illustrations and guidance regarding severity levels are given in Appendix C for rutting, as well as for other types of Type A damage, and the user should refer to that Appendix for complete details.

![Figure I.1 Rutting – Severity Level 2](image)
Description of Type B Damage

Similarly, damage of Type B is characterized by a severity level, as detailed in Appendix A. One example is that of cracking of longitudinal joints. This type of crack is initially single and straight (Severity 1, e.g. Figure B.2). Under traffic, the crack evolves, branches or doubles, and eventually loses material at the edges (Severity 2,3).

![Figure B.2 Cracking – Severity Level 1](image)

Rigid Pavements

As for flexible pavements, a visual condition (distress) survey is essential for planning rehabilitation efforts. In order to make knowledgeable decisions, the engineer should have the following information:

- Distress type
- Distress severity
- Distress amount

The distress type will assist in defining probable cause(s) of the distress and ultimately help in selecting a rehabilitation strategy suitable to repair and prevent the recurrence of the problem. The distress is not necessarily related to traffic loads. Tables B.1 and B.2 (from Ref. 3) classify the distress types of jointed concrete and continuously reinforced concrete pavements respectively.

The condition survey will also document the severity of the distress and provide a record of the pavement condition at the time of the survey, including the location of the distress. Thus, differences between sections of the pavement (or lanes) will become apparent and guide in the rehabilitation design process.

Definitions of the major distress types and of severity levels suggested for use during the condition survey are presented in Appendix D, together with recommended methods of measurement (to record the distress amount).
DRAINAGE SURVEY

In the design of pavement rehabilitation, the possible contribution of drainage improvement should be investigated. The existing drainage of the pavement and subgrade may be inadequate and the pavement distress may result from this condition or be accelerated by it.

The pavement condition survey will often indicate moisture-related distress. Distress types in flexible pavements which may either be caused or be accelerated by moisture include stripping, rutting, depression, fatigue cracking and potholes. In rigid pavement, they include pumping, “D” cracking, joint deterioration, faulting and corner breaks.

Subsequent to the pavement condition survey and depending on whether or not drainage-related distress is apparent, a determination needs to be made regarding the need for a specific drainage survey.

Table I.1    General Categorization of Jointed Concrete Pavement Distress
(from Ref. 3)

<table>
<thead>
<tr>
<th>Distress Type</th>
<th>Primarily Traffic Load Caused</th>
<th>Primarily Climate/Materials Caused</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Blow-up</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>2. Corner break</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>3. Depression</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>4. Durability “D” cracking</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>5. Faulting of transverse joints and cracks</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>6. Joint load transfer associated distress</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>7. Joint seal damage of transverse joints</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>8. Lane/shoulder dropoff or heave</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>9. Lane/shoulder joint separation</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>10. Longitudinal cracks</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>11. Longitudinal joint faulting</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>12. Patch deterioration</td>
<td>X (M, H)</td>
<td>X (L)</td>
</tr>
<tr>
<td>13. Patch adjacent slab deterioration</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>14. Popouts</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>15. Pumping and water bleeding</td>
<td>X (M, H)</td>
<td>X (L)</td>
</tr>
<tr>
<td>16. Reactive aggregate durability distress</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>17. Scaling, map cracking and crazing</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>18. Spalling (transverse and longitudinal joints)</td>
<td>X (M, H)</td>
<td>X (L, M, H)</td>
</tr>
<tr>
<td>19. Spalling (corner)</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>20. Swell</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>21. Transverse and diagonal cracks</td>
<td>X (L, M, H)</td>
<td>X (L)</td>
</tr>
</tbody>
</table>
Table I.2  General Categorization of Continuously Reinforced Concrete Pavement Distress (from Ref. 3)

<table>
<thead>
<tr>
<th>Distress Type</th>
<th>Primarily Traffic Load Caused</th>
<th>Primarily Climate/Materials Caused</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Asphalt patch deterioration</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>2. Blow-up</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>3. Concrete patch deterioration</td>
<td>X (M, H)</td>
<td>X (L)</td>
</tr>
<tr>
<td>4. Construction joint distress</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>5. Depression</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6. Durability “D” cracking</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>7. Edge punchout</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8. Lane/shoulder dropoff or heave</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>9. Lane/shoulder joint separation</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>10. Localized distress</td>
<td></td>
<td></td>
</tr>
<tr>
<td>11. Longitudinal cracking</td>
<td></td>
<td></td>
</tr>
<tr>
<td>12. Longitudinal joint faulting</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>13. Patch adjacent slab deterioration</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>14. Popouts</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15. Pumping and water bleeding</td>
<td>X (M, H)</td>
<td>X (L)</td>
</tr>
<tr>
<td>16. Reactive aggregate distress</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>17. Scaling, map cracking and crazing</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>18. Spalling</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>19. Swell</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20. Transverse cracking</td>
<td>X (M, H)</td>
<td>X (L, M)</td>
</tr>
</tbody>
</table>

Note: L, M, H refer to low, medium and high severity levels.

It should be recognized that even when moisture-related distress is absent, drainage deficiencies may exist which need to be corrected. Maintenance personnel regularly involved with a particular section of road under evaluation should be consulted in this regard, as they are a good source of relevant information.

The first step in drainage evaluation will include a review of the as-built documents collected as part of the data collection discussed in Chapter 2. That will encompass the determination of the initial drainage provisions, and examination of pavement cross-sections and profiles. If moisture-related distress is apparent, then the original drainage system is inadequate (due to design or present condition). The next step of the evaluation will include an examination of topographical features. This may lead to discovering streams or wet areas above the pavement elevation.

The drainage evaluation also requires a site investigation (preferably during a wet season), with the aim of answering such questions as the following:

- Where and how does water move across the pavement surface?
- Where does water collect on and near the pavement?
- How high is the water level in the ditches?
- Do the joints and cracks contain any water?
Does water pond on the shoulder?
Does water-loving vegetation flourish along the roadside?
Are deposits of fines or other evidence of pumping (blowholes) visible at the pavement’s edge?
Do the inlets contain debris or sediment buildup?
Are the joints and cracks sealed well?

If the original drainage design appears adequate, the site investigation should also verify that it has actually been entirely built as planned, and determine whether changes have taken place since it was built.

Drainage problems often encountered include shallow side ditches, broken or clogged pipes and outlets, permeable shoulders and slow draining aggregate roadbase with no outlets or outlets of insufficient capacity.

Subgrade problems related to drainage include pockets of poor soils (e.g. saturated silts and/or clays, or organic materials) and localized springs, groundwater seepage or non-functioning subsurface drainage systems.

**NDT DEFLECTION MEASUREMENTS**

The use of non-destructive testing (NDT) for the measurement of pavement deflection under applied loading has been part of the process of structural evaluation and rehabilitation design for decades, using various types of equipment (e.g. Ref. 13). The total deflection under load has been used as an indicator of the load-carrying capacity of the pavement, especially for flexible pavements, by correlating allowable repetitions of ESAs to the maximum deflection. This has later been refined by taking into consideration only the “rebound” or “elastic” portion of the deflection as the key indicator of performance, rather than the total deflection under load. Various methods utilizing measurements of the slope (e.g. radius of curvature) of the deflection basin (or deflection bowl) under load have also been developed to further characterize the pavement behavior and structural capacity.

In this manual, emphasis is placed on measurements of the rebound deflection made by means of a Benkelman Beam. This presents the advantages of a certain consistency with the procedures of the PMS, a relatively simple field procedure, and a consistency with the selected overlay design methods presented further in the manual. An alternate deflection survey procedure is also outlined, using a Falling Weight Deflectometer (FWD). This method, which is well-suited and actually intended to make use of deflection bowl measurements, requires more logistical support and is more expensive. It may be used with the explicit approval of RDA.

**Benkelman Beam Rebound Deflection Testing Procedure**

The following (adapted from Ref. 11) gives a procedure for determining the static Benkelman Beam rebound deflection at a point on an asphalt pavement structure under specified axle load, tire size, tire spacing and tire pressure. The absolute minimum spacing of sample locations should be 100 m, on either side (in either direction) of the road. Shorter intervals should be adopted if practical, and in areas of special concern (particular distress).
**Equipment**

Major equipment includes:

1. A Benkelman Beam as per AASHTO T256.
   
   Painting the beam white or aluminum will reduce temperature effects.

2. A 4.5 metric tonne truck as the reaction load. The vehicle shall have an 8.2 tonne load equally distributed on the two dual wheels of the rear axle. The clear distance between two tires of each dual shall be a minimum of 50 mm. The tires shall be 10.00 x 20, 12 ply, inflated to a pressure of 552 kPa. The use of tires with tubes and rib treads is recommended.

3. Tire pressure-measuring gauge.

4. Standard Iron-Constantan thermocouple wire and a temperature potentiometer (any other surface temperature measuring equipment that gives results comparable to the thermocouple potentiometer equipment may be used).

**Procedure**

1. The preselected point to be tested is located (usually only the outside lane is tested) and marked on the pavement. Points should be located 0.6 m from the pavement edge if the lane width is less than 3.35 m; 0.9 m from the pavement edge if the lane width is 3.35 m or more.

2. Center one set of dual wheels of the truck over the marked point. A location within 75 mm of the point is acceptable.

3. Insert the probe of the Benkelman Beam between the dual wheels and place on the selected sample point.

4. Remove the locking pin from the beam and adjust the front legs to permit approximately 13 mm travel of the dial gauge stem.

5. Start the buzzer on the beam and record the initial dial reading.

6. Immediately after recording the reading, drive the truck slowly forward 9 m or more.

7. Record the final dial reading. When the dial movement stops, stop the buzzer. The dial movement may resume after a short pause, but no more readings are necessary.

8. Measure the pavement surface temperature* in the following manner:

---

* One pavement surface temperature measurement an hour usually will be sufficient. Make the measurement at the location currently being tested for deflection.
a. At a point not less than 254 mm from the pavement’s edge, drive a small hole into the pavement 3 mm in diameter and 3 mm in depth.

b. Fill the hole with water or asphalt. Insert a thermocouple with the wire bent at right angles 5 mm from the end, 3 mm into the water or asphalt.

c. Read the temperature with a temperature potentiometer. At this time, also record the air temperature.

9. Check the truck tire pressure once a day and correct to the 552 kPa standard equipment, if necessary.

10. Determine the total thickness, to the nearest 25 mm of asphalt-bound components in the pavement structure. This may be done from construction records (cf. Chapter 2), by core drilling, or by a small excavation at the pavement’s edge. Determine also the types and general condition of materials in the remainder of the pavement structure, i.e., “granular roadbase, wet” or “granular subbase, saturated and contaminated with silt from subgrade”.

Calculations

Subtract the final dial reading from the initial dial reading and record. The total pavement rebound deflection is twice the dial movement during the test (two-to-one is the usual ratio for the Benkelman Beam; some models may be built with a different ratio).

Report

The report shall include the following:

Sample location
Total pavement rebound deflection
Pavement surface temperature
Air temperature
Asphalt pavement thickness

For rigid pavements, the deflection measurements using the Benkelman Beam are made along the edge of the concrete slab, according to the following guidelines essentially taken from those of the Asphalt Institute (Ref. 11):

For two-lane highways, the deflection measurements are made on the outside edge on both sides of the centerline. For divided highways, deflections should be measured on the outermost edge only. Additional deflection measurements should be made at corners, joints, cracks, and deteriorated pavement areas, to determine load-transfer capability. The total vertical movement in these areas is important, but the differential movement from one slab to another is more important, because it may indicate the need for undersealing. The differential movement is considered a measurement of the load-transfer capability of the slab. Load-transfer is defined as the ratio of the deflection on one side of the joint or crack to the deflection on the opposite side. This is illustrated in Figure B.3.
The deflection testing procedure for rigid pavements is as follows:

The Benkelman Beam should be positioned on the shoulder as shown in Figure B.4.

Measurements should be made on either side of the joint or crack to determine the parameters described in Figure B.3. They should be taken, to the extent practical, during the most severe environmental circumstances, e.g. at the end of a wet season. A suggested minimum spacing is 100 m, with more closely spaced sample locations where cut and fill sections alternate rapidly.
Falling Weight Deflectometer (FWD)

The Falling Weight Deflectometer (FWD) is classed as an impulse deflection device providing a non-destructive means of determining a pavement response to dynamic loading.

Various models and manufacturers (e.g. Dynatest, Foundation Mechanics, KUAB, etc.) exist, which follow the same principle and provide similar information. A typical arrangement is given schematically in Figure B.5.

![Image of Falling Weight Deflectometer](image)

**Figure I.5 Falling Weight Deflectometer**

Weights are raised to a predetermined height and allowed to fall on a specially-designed plate, transmitting an impulse force to the pavement. The shape of the load pulse obtained is somewhat similar to that obtained from a moving wheel load. Figure B.6 schematically illustrates a typical pavement structure under a FWD dynamic load. By varying the weights dropped and the height from which they are dropped, the force developed upon impulse can be varied, as can the range of wheel load effects simulated. For instance, a typical dynamic force of 40 kN (a generally recommended magnitude) applied to a 30 cm diameter disk by a dropping weight on the order of 60 kg is common.
Usually, six or seven geophones (sensors, or velocity transducers) record the deflections at offset distances from the center of load application. A typical configuration for flexible pavements is given in Figure B.7.

![Figure B.7](image)

**Figure I.6** Schematic of Stress Zone within Pavement Structure under the FWD Load (from Ref. 3)

![Figure I.7](image)

**Figure I.7** Deflection Sensors Configuration for Flexible Pavement Testing

The tests should be made in the outer wheelpath at intervals sufficient to adequately assess conditions. Intervals of 25 m to 250 m are typical. Areas that are seriously
deteriorated and obviously will require repairs should not be tested, or tested separately so as not to influence statistical analyses of the results.

For rigid pavements, special test configurations can be used to evaluate joint load transfer and for slab void detection. Some typical configurations are illustrated in Figure B.8.

![Figure B.8 Deflection Sensors Configuration (Rigid Pavements)](image)

**Figure B.8** Deflection Sensors Configuration (Rigid Pavements)

The results of deflection are best analyzed by computer software to perform statistical analyses and to back-calculate relevant pavement and subgrade support parameters. Since the particular equipment used and the software vary, they should be submitted for approval to RDA. For use with Dynatest and KUAB FWDs, a companion software package to the AASHTO Guide (Ref. 3) is available (Ref. 14). The Guide also gives equations which can be used, but only on a point-by-point basis, and are not practical for use unless computerized (e.g., by spreadsheet programs).

**EXISTING PAVEMENT STRUCTURE AND SUBGRADE ASSESSMENT**

The data obtained from PMS records and the subsequent review of those records should serve as the basis for the fieldwork. The PMS data can be reconfirmed by carrying out a specific field study and reviewing the field data.

In general, field testing is categorized into two broad areas: nondestructive testing (NDT) and destructive testing. Destructive tests (such as test pits, DCP tests) require the physical removal of pavement layer material in order to obtain a sample (either disturbed or undisturbed) or to conduct an in-situ test. Such testing has many disadvantages and
limitations, particularly when conducted on moderate to heavily trafficked roadways. Practical restraints in terms of time and money severely limit the number and variety of destructive tests conducted on routine rehabilitation studies.

Nondestructive testing, on the other hand, does not necessitate physical disturbance of the pavement. The most widely used form of NDT is associated with the field deflection tests noted in Section B.3.

In spite of their disadvantages, destructive tests are the only definitive means of determining pavement layer thicknesses and layer material type.

Test Pits

Test pits represent one of the common methods of investigation to determine the thickness and type of the various pavement layers and to assess the subgrade. Samples from each pavement layer and subgrade can be collected for visual inspection and subsequent laboratory testing. The test results can be used in the rehabilitation design analysis and to check conformance of the material with standard specifications. Test pits shall be dug through the pavement layers and into the subgrade soil for a minimum total depth of 0.80 m (alternatively, the pit should extend at least 0.20 m below subgrade level).

The spacing of the test pits should depend on sound engineering judgement and be guided by a prior review of all possible documents, as well as a visual pavement condition survey.

However, as a general guideline, one test pit every 500 meters, alternating on either side of the roadway, is recommended. The position of each test pit shall be accurately determined and reported.

DCP Testing

DCP testing frequency can be the same as that of the test pits, i.e. every 500 m. In fact, the DCP test location can be adjacent to the test pit location. However, as mentioned before, depending on the pavement location, the frequency can be altered to better assess the pavement and to optimize the testing program.

The DCP instrument is designed for the rapid in-situ measurement of the structural properties of existing road pavements (with the exception of hard layers like concrete) and subgrade. In cases of material that the DCP cannot penetrate, a hole can be drilled through that layer using a suitable drill. Lower layers of the pavement can then be tested in the normal way.

The DCP test results can be compared with the laboratory test results, and the data collected from review of the possible documents, including PMS records. Additional testing may be necessary in case of discrepancies to better evaluate pavement conditions.
APPENDIX J

TYPICAL RIGID PAVEMENT DISTRESS
TYPE-SEVERITY DESCRIPTIONS

Name of Distress: Blow-up

Description:

Most blow-ups occur during the spring and hot summer at a transverse joint or wide crack. Infiltration of incompressible materials into the joint or crack during cold periods results in high compressive stresses in hot periods. When this compressive pressure becomes too great, a localized upward movement of the slab or shattering occurs at the joint or crack. Blow-ups are accelerated due to a spalling away of the slab at the bottom, creating reduced joint contact area. The presence of “D” cracking or freeze-thaw damage also weakens the concrete near the joint, resulting in increased spalling and blow-up potential.

Severity Levels:

- L—Blow-up has occurred, but only causes some bounce of the vehicle which creates no discomfort.
- M—Blow-up causes a significant bounce of the vehicle which creates some discomfort. Temporary patching may have been placed because of the blow-up.
- H—Blow-up causes excessive bounce of the vehicle, which creates substantial discomfort and/or a safety hazard and/or vehicle damage, requiring a reduction

HOW TO MEASURE:

Blow-ups are measured by counting the number existing in each uniform section. Severity level is determined by riding in a mid- to full-sized sedan weighing approximately 13.3-16.9 kN over the uniform section at the posted speed limit. The number is not as important as the fact that initial blow-ups signal a problem with “lengthening” or gradual downhill movement—and others should be expected to occur until the maximum distance is down to 300 meters between blow-ups, the distance required to develop full restraint of an interior section.

Name of Distress: Corner Break

Description:

A corner break is a crack that intersects the joints at a distance less than 1.8 m on each side, measured from the corner of the slab. A corner break extends vertically through the entire slab thickness. It should not be confused with a corner spall, which intersects the joint at an angle through the slab and is
Typically within 0.3 m from the slab corner. Heavy repeated loads, combined with pumping, poor load transfer across the joint, and thermal curling and moisture warping stresses, result in corner breaks.

Severity Levels:

L—Crack is tight (hairline). Well-sealed cracks are considered tight. No faulting or break-up of broken corner exists. Crack is not spalled.

M—Crack is working and spalled at medium severity, but break-up of broken corner has not occurred. Faulting of crack or joint is less than 13 mm. Temporary patching may have been placed because of corner break.

H—Crack is spalled at high severity, the corner piece has broken into two or more pieces, or faulting of crack or joint is more than 13 mm.

How to Measure:

Corner breaks are measured by counting the number that exists in the uniform section. Different levels of severity should be counted and recorded separately. Corner breaks adjacent to a patch will be counted as “Patch adjacent slab deterioration”.

Name of Distress: Depression

Description:

Depressions in concrete pavements are localized settled areas. There is generally significant slab cracking in these areas, due to uneven settlement. The depressions can be located by stains caused by oil droppings from vehicles and by riding over the pavement. Depressions can be caused by settlement or consolidation of the foundation soil, or can be “built-in” during construction. They are frequently found near culverts. This is usually caused by poor compaction of soil around the culvert during construction. Depressions cause slab cracking, roughness, and hydroplaning when filled with water of sufficient depth.

Severity Levels:

L—Depression causes a distinct bounce of vehicle which creates no discomfort.

M—Depression causes significant bounce of the vehicle, which creates some discomfort.

H—Depression causes excessive bounce of the vehicle, which creates substantial discomfort, and/or a safety hazard, and/or vehicle damage, requiring a reduction in speed for safety.

How to Measure:

Depressions are measured by counting the number that exists in each uniform section. Each depression is rated according to its level of severity. Severity level is determined by riding in a mid- to full-sized sedan weighing approximately 13.3-16.9 kN over the uniform section at the posted speed limit.
Name of Distress: Durability (“D”) Cracking

Description:

“D” cracking is a series of closely-spaced, crescent-shaped hairline cracks that appear at a PCC pavement slab surface adjacent and roughly parallel to transverse and longitudinal joints, transverse and longitudinal cracks, and the free edges of pavement slab. The fine surface cracks often curve around the intersection of longitudinal joints/cracks and transverse joints/cracks. These surface cracks often contain calcium hydroxide residue, which causes a dark coloring of the crack and immediate surrounding area. This may eventually lead to disintegration of the concrete within 0.3 to 0.6 m or more of the joint or crack, particularly in the wheelpaths. “D” cracking is caused by freeze-thaw expansive pressures of certain types of coarse aggregates and typically begins at the bottom of the slab, which disintegrates first. Concrete durability problems caused by reactive aggregates are rated under “Reactive Aggregate Distress”.

Severity Levels:

L—The characteristic pattern of closely-spaced fine cracks has developed near joints, cracks, and/or free edges; however, the width of the affected area is generally <30 cm wide at the center of the lane in transverse cracks and joints. The crack pattern may fan out at the intersection of transverse cracks/joints with longitudinal cracks/joints. No joint/crack spalling has occurred, and no patches have been placed for “D” cracking.

M—The characteristic pattern of closely-spaced cracks has developed near the crack, joint, or free edge and: (1) is generally wider than 30 cm at the center of the lane in transverse cracks and/or joints; or (2) low- or medium-severity joint/crack or corner spalling has developed in the affected area; or (3) temporary patches have been placed due to “D” cracking-induced spalling.

H—The pattern of fine cracks has developed near joints or cracks and (1) a high severity level of spalling at joints/cracks exists and considerable material is loose in the affected area; or (2) the crack pattern has developed generally over the entire slab area between cracks and/or joints.

How to Measure:

“D” cracking is measured by counting the number of joints or cracks (including longitudinal) affected. Different severity levels are counted and recorded separately. “D” cracking adjacent to a patch is rated as patch-adjacent slab deterioration. “D” cracking should not be counted if the fine crack pattern has not developed near cracks, joints, and free edges. Popouts and discoloration of joints, cracks, and free edges may occur without “D” cracking.
Name of Distress: Faulting of Transverse Joints and Cracks

Description:

Faulting is the difference of elevation across a joint or crack. Faulting is caused in part by a buildup of loose materials under the approach slab near the joint or crack, as well as depression of the leave slab. The buildup of eroded or infiltrated materials is caused by pumping from under the leave slab and shoulder (free moisture under pressure) due to heavy loadings. The warp and/or curl upward of the slab near the joint or crack due to moisture and/or temperature gradient contributes to the pumping condition. Lack of load transfer contributes greatly to faulting.

Severity Levels:

Severity is determined by the average faulting over the joints within the sample unit.

How to Measure:

Faulting is determined by measuring the difference in elevation of slabs at transverse joints for the slabs in the sample unit. Faulting of cracks is measured as a guide to determine the distress level of the crack. Faulting is measured 30 cm in from the outside (right) slab edge on all lanes except the innermost passing lane. Faulting is measured 30 cm in from the inside (left) slab edge on the inner passing lane. If temporary patching prevents measurement, proceed on to the next joint. Sign convention: + when approach slab is higher than departure slab, – when the opposite occurs. Faulting never occurs in the opposite direction.

Name of Distress: Joint Load Transfer System Associated Deterioration (Second Stage Cracking)

Description:

This distress develops as a transverse crack a short distance (e.g., 23 cm) from a transverse joint at the end of joint dowels. This usually occurs when the dowel system fails to function properly due to extensive corrosion or misalignment. It may also be caused by a combination of smaller diameter dowels and heavy traffic loadings.

Severity Levels:

L—Hairline (tight) crack with no spalling or faulting or well-sealed crack with no visible faulting or spalling.

M—Any of the following conditions exist: the crack has opened to a width less than 25 mm; the crack has faulted less than 13 mm; the crack may have spalled to a low- or medium-severity level; the area between the crack and joint has started to break up, but pieces have not been dislodged to the point that a tire damage or safety hazard is present; or temporary patches have been placed due to this joint deterioration.
H—Any of the following conditions exist: a crack with width of opening greater than 25 mm; a crack with a high-severity level of spalling; a crack faulted 13 mm or more; or the area between the crack and joint has broken up and pieces have been dislodged to the point that a tire damage or safety hazard is present.

How to Measure:
The number of joints with each severity level are counted in the uniform section.

Name of Distress: Joint Seal Damage of Transverse Joints

Description:
Joint seal damage exists when incompressible materials and/or water can infiltrate into the joints. This infiltration can result in pumping, spalling, and blow-ups. A joint sealant bonded to the edges of the slabs protects the joints from accumulation of incompressible materials and also reduces the amount of water seeping into the pavement structure. Typical types of joint seal damage are: (1) stripping of joint sealant, (2) extrusion of joint sealant, (3) weed growth, (4) hardening of the filler (oxidation), (5) loss of bond to the slab edges, and (6) lack or absence of sealant in the joint.

Severity Levels:

L—Joint sealant is in good condition throughout the section with only a minor amount of any of the above types of damage present. Little water and no incompressibles can infiltrate through the joint.

M—Joint sealant is in fair condition over the entire surveyed section, with one or more of the above types of damage occurring to a moderate degree. Water can infiltrate the joint fairly easily; some incompressibles can infiltrate the joint. Sealant needs replacement within 1 to 3 years.

H—Joint sealant is in poor condition over most of the sample unit, with one or more of the above types of damage occurring to a severe degree. Water and incompressibles can freely infiltrate the joint. Sealant needs immediate replacement.

How to Measure:
Joint sealant damage of transverse joints is rated based on the overall condition of the sealant over the entire sample unit.

Name of Distress: Lane/Shoulder Drop-Off or Heave

Description:
Lane/shoulder drop-off or heave occurs when there is a difference in elevation between the traffic lane and shoulder. Typically, the outside shoulder settles due to consolidation or a settlement of the underlying granular or subgrade material or pumping of the underlying material. Heave of the shoulder may occur due to frost action or swelling.
soils. Drop-off of granular or soil shoulder is generally caused from blowing away of shoulder material from passing trucks.

**Severity Levels:**

Severity level is determined by computing the mean difference in elevation between the traffic lane and shoulder.

**How to Measure:**

Lane/shoulder drop-off or heave is measured in the sample unit at all joints when joint spacing is > 15 m, at every third joint when spacing is < 15 m. It is also measured at mid-slab in each slab measured at the joint. The mean difference in elevation is computed from the data and used to determine severity level. Measurements at joints are made 0.3 m from the transverse joint on the departure slab only on the outer lane/shoulder.

**Name of Distress: Lane/Shoulder Joint Separation**

**Description:**

Lane/shoulder joint separation is the widening of the joint between the traffic lane and the shoulder, generally due to movement in the shoulder. If the joint is tightly closed or well-sealed so that water cannot easily infiltrate, then lane/shoulder joint separation is not considered a distress.

**Severity Levels:**

No severity level is recorded if the joint is tightly sealed.

- **L**—Some opening, but less than or equal to 3 mm.
- **M**—More than 3 mm, but equal or less than 10 mm opening.
- **H**—More than 10 mm opening. Gravel or sod shoulders are rated as high.

**How to Measure:**

Lane/shoulder joint separation is measured and recorded in mm near transverse joints and at mid-slab. The mean separation is used to determine the severity level.

**Name of Distress: Longitudinal Cracks**

**Description:**

Longitudinal cracks occur generally parallel to the centerline of the pavement. They are often caused by improper construction of longitudinal joints or by a combination of heavy load repetition, loss of foundation support, and thermal and moisture gradient stresses.
Severity Levels:

L—Hairline (tight) crack with no spalling or faulting, or a well-sealed crack with no visible faulting or spalling.
M—Working crack with a moderate or less severity spalling and/or faulting less than 12 mm.
H—A crack with width greater than 25 mm; a crack with a high-severity level of spalling; or a crack faulted 13 mm or more.

How to Measure:

Cracks are measured in meters for each level of distress. The length and average severity of each crack should be identified and recorded.

Name of Distress: Longitudinal Joint Faulting

Description:

Longitudinal joint faulting is a difference in elevation of two traffic lanes measured at the longitudinal joint. It is caused primarily by heavy truck traffic and settlement of the foundation.

Severity Levels:

Severity level is determined by measuring the maximum fault.

How to Measure:

Where the longitudinal joint has faulted, the length of the affected area and the maximum joint faulting is recorded.

Name of Distress: Patch Deterioration (including replaced slabs)

Description:

A patch is an area where a portion or all of the original slab has been removed and replaced with a permanent type of material (e.g., concrete or hot-mixed asphalt). Only permanent patches should be considered.

Severity Levels:

L—Patch has little or no deterioration. Some low severity spalling of the patch edges may exist. Faulting across the slab-patch joints must be less than 6 mm. Patch is rated low severity even if it is in excellent condition.
M—Patch has cracked (low severity level) and/or some spalling of medium-severity level exists around the edges. Minor rutting may be present. Faulting of 6-19 mm
exists. Temporary patches may have been placed because of permanent patch deterioration.

H—Patch has deteriorated by spalling, rutting, or cracking within the patch to a condition which requires replacement.

How to Measure:

The number of patches within each uniform section is recorded. Patches at different severity levels are counted and recorded separately. Additionally, the approximate area (in square meters) of each patch and type (i.e., PCC or asphalt) is recorded. All patches are rated either L, M, or H.

Name of Distress: Patch Adjacent Slab Deterioration

Description:

Deterioration of the original concrete slab adjacent to a permanent patch is given the above name. This may be in the form of spalling of the slab at the slab/patch joint, “D” cracking of the slab adjacent to the patch, a corner break in the adjacent slab, or a second permanent patch placed adjacent to the original patch.

Severity Levels:

Severity levels are the same as that described for the particular distress found. A second permanent patch, placed adjacent to a previously-placed permanent patch, will be rated here as medium severity. Temporary patches placed because of this deterioration will also be rated here as medium severity.

How to Measure:

The number of permanent patches with distress in the original slab adjacent to the patch at each severity level will be counted and recorded separately. Additionally, the type of patch (AC or PCC) and distress will be recorded separately.

Name of Distress: Popouts

Description:

A popout is a small piece of concrete that breaks loose from the surface due to freeze-thaw action, expansive aggregates, and/or nondurable materials. Popouts may be indicative of unsound aggregates and “D” cracking. Popouts typically range from approximately 25 mm to 10 cm in diameter and from 13 to 51 mm deep.

Severity Levels:

No degrees of severity are defined for popouts. The average popout density must exceed approximately one popout per square meter over the entire slab area before they are counted as a distress.
How to Measure:

The density of popouts can be determined by counting the number of popouts per square meter of surface in areas having typical amounts.

Name of Distress: Pumping and Water Bleeding:

Description:

Pumping is the movement of material by water pressure beneath the slab when it is deflected under a heavy moving wheel load. Sometimes the pumped material moves around beneath the slab, but often it is ejected through joints and/or cracks (particularly along the longitudinal lane/shoulder joint with an asphalt shoulder). Beneath the slab there is typically particle movement counter to the direction of traffic across a joint or crack that results in a buildup of loose materials under the approach slab near the joint or crack. Many times, some fine materials are pumped out, leaving a thin layer of relatively loose clean sand and gravel beneath the slab, along with voids causing loss of support. Pumping occurs even in pavement sections containing stabilized subbases.

Water bleeding occurs when water seeps out of joints and/or cracks. Many times it drains out over the shoulder in low areas.

Severity Levels:

L—Water is forced out of a joint or crack when trucks pass over the joints or cracks; water is forced out of the lane/shoulder longitudinal joint when trucks pass along the joint; or water bleeding exists. No fines can be seen on the surface of the traffic lanes or shoulder.

M—A small amount of pumped material can be observed near some of the joints or cracks on the surface on the traffic lane or shoulder. Blow holes may exist.

H—A significant amount of pumped materials exist on the pavement surface of the traffic lane or shoulder along the joints or cracks.

How to Measure:

If pumping or water bleeding exists anywhere in the sample unit, it is counted as occurring at highest severity level, as defined above.

Name of Distress: Reactive Aggregate Distresses

Description:

Reactive aggregates either expand in alkaline environments or develop prominent siliceous reaction rims in concrete. It may be an alkali-silica reaction or an alkali-carbonate reaction. As expansion occurs, the cement matrix is disrupted and cracks. It appears as a map-cracked area; however, the cracks may go deeper into the concrete than
in normal map cracking. It may affect most of the slab or it may first appear at joints and cracks.

**Severity Levels:**

Only one level of severity is defined. If alkali-aggregate cracking occurs anywhere in the slab, it is counted. If the reaction has caused spalling or map cracking, these are also counted.

**How to Measure:**

Reactive-aggregate distress is measured in square feet or square meters.

**Name of Distress: Scaling and Map Cracking or Crazing**

**Description:**

Scaling is the deterioration of the upper 3 to 13 cm of the concrete slab surface. Map cracking or crazing is a series of fine cracks that extend only into the upper surface of the slab surface. Map cracking or crazing is usually caused by overfinishing of the slab and may lead to scaling of the surface. Scaling can also be caused by reinforcing steel being too close to the surface.

**Severity Levels:**

- **L**—Crazing or map cracking exists; the surface is in good condition with no scaling.
- **M/H**—Scaling exists.

**How to Measure:**

Scaling and map cracking or crazing are measured by area of slab in square meters.

**Name of Distress: Spalling (Transverse and Longitudinal Joint/Crack):**

**Description:**

Spalling of cracks and joints is the cracking, breaking or chipping (or fraying) of the slab edges within 0.6 m of the joint/crack. A spall usually does not extend vertically through the whole slab thickness but extends to intersect the joint at an angle. Spalling usually results from (1) excessive stresses at the joint or crack caused by infiltration of incompressible materials and subsequent expansion, (2) disintegration of the concrete from freeze-thaw action of “D” cracking, (3) weak concrete at the joint (caused by honeycombing), (4) poorly designed or constructed load transfer device (misalignment, corrosion), and/or (5) heavy repeated traffic loads.

**Severity Levels:**

- **L**—The spall or fray does not extend more than 8 cm on either side of the joint or crack. No temporary patching has been placed to repair the spall.
M—The spall or fray extends more than 8 cm on either side of the joint or crack. Some pieces may be loose and/or missing, but the spalled area does not present a tire damage or safety hazard. Temporary patching may have been placed because of spalling.

H—The joint is severely spalled or frayed to the extent that a tire damage or safety hazard exists.

How to Measure:

Spalling is measured by counting and recording separately the number of joints with each severity level. If more than one level of severity exists along a joint, it will be recorded as containing the highest severity level present. Although the definition and severity levels are the same, spalling of cracks should not be recorded. The spalling of cracks is included in rating severity levels of cracks. Spalling of transverse and longitudinal joints will be recorded separately. Spalling of the slab edge adjacent to a permanent patch will be recorded as patch adjacent slab deterioration. If spalling is caused by “D” cracking, it is counted as both spalling and “D” cracking at appropriate severity levels.

Name of Distress: Spalling (Corner):

Description:

Corner spalling is the raveling or breakdown of the slab within approximately 0.3 m of the corner. However, corner spalls with both edges less than 8 cm long will not be recorded. A corner spall differs from a corner break in that the spall usually angles downward at about 45° to intersect the joint, while a break extends vertically through the slab. Corner spalling can be caused by freeze-thaw deterioration, “D” cracking, and other factors.

Severity Levels:

<table>
<thead>
<tr>
<th>Level</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>L</td>
<td>Spall is not broken into pieces and not loose.</td>
</tr>
<tr>
<td>M</td>
<td>One of the following conditions exists: Spall is broken into pieces; cracks are spalled; some or all pieces are loose or absent but do not present tire damage or safety hazard; or spall is patched.</td>
</tr>
<tr>
<td>H</td>
<td>Pieces of the spall are missing to the extent that the hole presents a tire damage or safety hazard.</td>
</tr>
</tbody>
</table>

How to Measure:

Corner spalling is measured by counting and recording separately the number of corners spalled at each severity level within the sample unit.
Name of Distress: Swell

Description:

A swell is an upward movement or heave of the slab surface, resulting in a sometimes sharp wave. The swell is usually accompanied by slab cracking. It is usually caused by frost heave in the subgrade or by an expansive soil. Swells can often be identified by oil droppings on the surface as well as riding over the pavement in a vehicle.

Severity Levels:

L—Swell causes distinct bounce of the vehicle which causes no discomfort.
M—Swell causes significant bounce of the vehicle which creates some discomfort.
H—Swell causes excessive bounce of the vehicle which creates substantial discomfort, and/or a safety hazard, and/or vehicle damage, requiring a reduction in speed for safety.

How to Measure:

The number of swells within the uniform section are counted and recorded by severity level. Severity levels are determined by riding in a mid- to full-sized sedan weighing approximately 13.3-16.9 kN over the uniform section at the posted speed limit.

Name of Distress: Transverse and Diagonal Cracks

Description:

Linear cracks are caused by one or a combination of the following: heavy load repetition, thermal and moisture gradient stresses, and drying shrinkage stresses. Medium- or high-severity cracks are working cracks and are considered major structural distresses. They may sometimes be due to deep-seated differential settlement problems. (Note: Hairline cracks that are less than 1.8 m long are not rated.)

Severity Levels:

L—Hairline (tight) crack with no spalling or faulting, a well-sealed crack with no visible faulting or spalling.
M—Working crack with low- to medium-severity level of spalling, and/or faulting less than 13 mm. Temporary patching may be present.
H—A crack with width of greater than 25 mm; a crack with a high-severity level of spalling; or a crack faulted 13 mm or more.

How to Measure:

The number and severity level of each crack should be identified and recorded. If the crack does not have the same severity level along the entire length, the crack is rated at the highest severity level present. Cracks in patches are recorded under patch deterioration.
APPENDIX K
METHOD FOR PREDICTING MEAN PAVEMENT TEMPERATURE (adapted from Ref. 1)

Definition and Purpose:

Mean pavement temperature is an average of the temperatures at the surface, mid-depth, and bottom of the asphalt-bound portion of the pavement, even though it may have been placed in several layers and at different times.

The mean pavement temperature is used to find a Temperature Adjustment Factor, required for adjusting pavement deflection values to a standard temperature of 21°C, as indicated in Step 1 of Appendix B.2 of the manual.

Information Required:

The temperature prediction method requires five items of information:

1. Location of test site—for identification of test data to guide in selection of the weather station from which air temperature data must be obtained.

2. Date of test—required to indicate the dates on which air temperature will be needed for pavement temperature adjustment.

3. Maximum and minimum air temperature—needed for each of the five days immediately prior to the date of deflection testing to provide an air-temperature history at the test site.

4. Pavement surface temperature—measured at the time the deflection test is performed.

5. Thickness of asphalt-bound portion of the pavement—required for selection of the proper curves on the pavement temperature chart.

All information listed above, except Item 3, is recorded during performance of the Benkelman beam deflection survey.

Item 3, the five days of air temperature history, prior to the date of the deflection measurements, can be obtained in one of three ways.

1. Hourly air temperatures may be read and recorded for a period of five days preceding each test location in the general vicinity where deflection measurements are made. From these data the maximum and minimum daily temperatures are determined.

2. Obtain from the nearest weather station the maximum and minimum air temperatures for each of the five days preceding each day of testing. This station should be in a location that records essentially the same temperature readings as at the test site.

3. Obtain the maximum and minimum air temperatures for each of the five days preceding each day of testing from published weather bureau (or similar agency) data. This is the most practical approach if the evaluation of test results is needed immediately.
When the five-day air temperature history has been obtained, the average of the ten values is determined. This average is used, together with pavement surface temperature to estimate the pavement temperature at any depth. Surface temperature is measured and recorded when the deflection test is performed. The date and time of these are also recorded.

Surface temperature may be measured by inserting a thermometer or thermocouple into a small, 6mm± deep hole or indentation in the surface of the pavement. The hole should be filled with water or asphalt, which should be allowed to come to equilibrium before the temperature is read.

**Prediction of Mean Pavement Temperature:**

Relationships between pavement surface temperature, air-temperature history, and pavement temperature at any depth have been derived and are shown in condensed form in Figure D-1. It should be used for overlay design only. Pavement temperatures at various depths can be estimated by applying the required information to this chart.

The first step is to add the five-day air temperature history (average of maximum and minimum air temperatures for five days preceding the test) and the pavement surface temperature to determine the value with which to enter the chart. Then, the *Mean Pavement Temperature* is estimated:

(a) Determine pavement thickness and mid-depth.

(b) Enter Figure D-1 on the horizontal scale with the pavement surface temperature. Extend a line vertically to intersect the depth lines. Extend lines horizontally from the depth representing the bottom of the layer to intersect the vertical scale. Interpolate between the depth lines as necessary.

(c) The sum of the surface temperature, mid-depth temperature, and bottom temperature is averaged to provide the Mean Pavement Temperature.

(d) The Mean Pavement Temperature is used to enter Figure 4.1 in the Manual to obtain a temperature adjustment factor for Benkelman beam deflection readings.
Example

Given: Asphalt concrete pavement 10cm thick. Surface temperature is 31°C. Average air temperature for 5 days preceding the test is 22°C. The adjusted surface temperature is 31 + 22 = 53°C (surface temperature plus 5-day average air temperature). Find predicted pavement mean temperature.

1. Surface temperature = 31°C
2. Temperature at 5cm depth = 30°C (From Figure D-1)
3. Temperature at 10cm depth = 27°C (From Figure D-1)
4. Pavement mean temperature =
   \[
   \frac{31° + 30° + 27°}{3} = 29°
   \]

(Use this temperature in Figure 4.1 of the manual to find temperature adjustment factor, F.)
APPENDIX L
DERIVATION OF SN\textsubscript{new}, STRUCTURAL NUMBER OF A NEW PAVEMENT

SN\textsubscript{new} is required in the component analysis procedure (Section 4.1 of the manual) to determine a required asphalt overlay thickness. SN\textsubscript{new} is computed in three steps as follows:

- **Step 1:** Select an appropriate required structure of a new pavement for the specific subgrade strength and traffic applicable to the project, in accordance with the procedure detailed in RDA’s Pavement Design Manual-2014. The structure selected is characterized by the thicknesses $T_i$ of its component layers, i.e. $T_1$, $T_2$, $T_3$= thicknesses of required pavement surfacing, roadbase and subbase layers respectively.

- **Step 2:** To each of the layers determined in Step 1, assign an appropriate structural layer coefficient $a_i$.

  The following structural layer coefficients are recommended:
  - Bituminous surface: $a_1 = 0.44$
  - Bituminous roadbase: $a_1 = 0.30$ (note: use 0.25 for in-place recycled materials)
  - Cement or lime stabilized roadbase: $a_2 = 0.15$ to 0.20
  - Granular roadbase: $a_2 = 0.14$
  - Cement or lime stabilized subbase: $a_3 = 0.12$
  - Granular subbase: $a_3 = 0.11$
  - Granular capping layer: $a_3 = 0.09$

- **Step 3:** compute SN\textsubscript{new} as:

$$SN_{\text{new}} = a_1T_1 + a_2T_2 + a_3T_3$$

*Example*

The example of Section 4.1 is considered: subgrade strength class S4, anticipated future traffic class T6.

- **Step 1:**

  An adequate new pavement structure consists of:
  - 10 cm AC surfacing
  - 20 cm granular roadbase
  - 17.5 cm subbase

- **Step 2:**

  Structural layer coefficients are assigned as follows: $a_1 = 0.44$; $a_2 = 0.14$; $a_3 = 0.11$

- **Step 3:**

  $$SN_{\text{new}} = a_1T_1 + a_2T_2 + a_3T_3 = 0.44 \times 10 + 0.14 \times 20 + 0.11 \times 17.5 = 9.13$$
APPENDIX M

DERIVATION OF $S\text{N}_{\text{eff}}$, EFFECTIVE STRUCTURAL NUMBER OF AN EXISTING PAVEMENT
(adapted from Ref. 3)

The NDT method of $S\text{N}_{\text{eff}}$ determination follows an assumption that the structural capacity of the pavement is a function of its total thickness and overall stiffness. The relationship between $S\text{N}_{\text{eff}}$ and stiffness is:

$$SN_{eff} = \frac{6T_t}{1,000} \sqrt[3]{E_p}$$  \hspace{1cm} (Equation 1)

where $T_t$ = total thickness (in centimeters) of all pavement layers above the subgrade

$E_p$ = effective modulus of pavement layers above the subgrade (kPa)

$E_p$ may be back-calculated from deflection data as follows.

The data required for the calculations is:

- $d_o$ = deflection measured at the center of the load plate (and adjusted to a standard temperature of 20° C), cm
- $P$ = applied load, kg
- $D$ = total thickness of pavement layers above the subgrade, cm
- $d_r$ = deflection at a distance $r$ from the center of the load, cm
- $r$ = distance from center of load, cm
- $a$ = NDT load plate radius, cm

With the above data, using Figure F-1, the ratio of $E_p/M_R$ can be calculated.

Where $M_R$ = subgrade resilient modulus, kPa

$$= \frac{23.54P}{d_r}$$

Based on the ratio of $E_p/M_R$ and known value of $M_R$, $E_p$ can be computed in kPa. $S\text{N}_{\text{eff}}$ is then calculated using Equation 1.
Figure M.1  Determination of $E_p/M_R$ (Ref. 3)
# APPENDIX N

## DEFINITIONS

<table>
<thead>
<tr>
<th>Term</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aggregate</td>
<td>Hard mineral elements of construction material mixtures, for example: sand, gravel (crushed or uncrushed) or crushed rock.</td>
</tr>
<tr>
<td>Asphalt Concrete</td>
<td>A mixture to predetermined proportions of aggregate, filler and bituminous binder material plant mix and usually placed by means of a paving machine.</td>
</tr>
<tr>
<td>Asphalt Surfacing</td>
<td>The layer or layers of asphalt concrete constructed on top of the roadbase, and, in some cases, the shoulders.</td>
</tr>
<tr>
<td>Average Annual Daily Traffic (AADT)</td>
<td>The total yearly traffic volume in both directions divided by the number of days in the year.</td>
</tr>
<tr>
<td>Average Daily Traffic (ADT)</td>
<td>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.</td>
</tr>
<tr>
<td>Base Course</td>
<td>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.</td>
</tr>
<tr>
<td>Binder Course</td>
<td>The lower course of an asphalt surfacing laid in more than one course.</td>
</tr>
<tr>
<td>Borrow Area</td>
<td>An area within designated boundaries, approved for the purpose of obtaining borrow material. A borrow pit is the excavated pit in a borrow area.</td>
</tr>
<tr>
<td>Borrow Material</td>
<td>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.</td>
</tr>
<tr>
<td>Boulder</td>
<td>A rock fragment, usually rounded by weathering or abrasion, with an average dimension of 0.30 m or more.</td>
</tr>
<tr>
<td>Bound Pavement Materials</td>
<td>Pavement materials held together by an adhesive bound between the materials and another binding material such as bitumen.</td>
</tr>
<tr>
<td>Camber</td>
<td>The convexity given to the curved cross-section of a roadway.</td>
</tr>
<tr>
<td>Capping Layer</td>
<td>(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.</td>
</tr>
<tr>
<td>Carriageway</td>
<td>That portion of the roadway including the various traffic lanes and auxiliary lanes but excluding shoulders.</td>
</tr>
<tr>
<td>Cross-Section</td>
<td>A vertical section showing the elevation of the existing ground, ground data and recommended works, usually at right angles to the centerline.</td>
</tr>
<tr>
<td>Term</td>
<td>Definition</td>
</tr>
<tr>
<td>--------------------------</td>
<td>-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Crossfall</td>
<td>The difference in level measured transversely across the surface of the roadway.</td>
</tr>
<tr>
<td>Culvert</td>
<td>A structure, other than a bridge, which provides an opening under the carriageway or median for drainage or other purposes.</td>
</tr>
<tr>
<td>Cutting</td>
<td>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.</td>
</tr>
<tr>
<td>Chippings</td>
<td>Stones used for surface dressing (treatment).</td>
</tr>
<tr>
<td>Design Period</td>
<td>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.</td>
</tr>
<tr>
<td>Diverted Traffic</td>
<td>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.</td>
</tr>
<tr>
<td>Equivalency Factors</td>
<td>Used to convert traffic volumes into cumulative standard axle loads.</td>
</tr>
<tr>
<td>Equivalent Single Axle Load (ESA)</td>
<td>Summation of equivalent 8.16 ton single axle loads used to combine mixed traffic to design traffic for the design period.</td>
</tr>
<tr>
<td>Fill</td>
<td>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.</td>
</tr>
<tr>
<td>Flexible Pavements</td>
<td>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.</td>
</tr>
<tr>
<td>Formation Level</td>
<td>Level at top of subgrade.</td>
</tr>
<tr>
<td>Generated Traffic</td>
<td>Additional traffic which occurs in response to the provision of improvement of the road.</td>
</tr>
<tr>
<td>Grading Modulus (GM)</td>
<td>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.</td>
</tr>
<tr>
<td>Heavy Vehicles</td>
<td>Those having an unloaded weight of 3000 kg or more.</td>
</tr>
<tr>
<td>Maintenance</td>
<td>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.</td>
</tr>
<tr>
<td>Mountainous Terrain</td>
<td>Terrain that is rugged and very hilly with substantial restrictions in both (terrain) horizontal and vertical alignment.</td>
</tr>
<tr>
<td>Normal Traffic</td>
<td>Traffic which would pass along the existing road or track even if no new pavement were provided.</td>
</tr>
<tr>
<td>Overlay</td>
<td>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.</td>
</tr>
<tr>
<td>Pavement Layers</td>
<td>The layers of different materials which comprise the pavement structure.</td>
</tr>
<tr>
<td>Project Specifications</td>
<td>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</td>
</tr>
<tr>
<td>Term</td>
<td>Definition</td>
</tr>
<tr>
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</tr>
<tr>
<td>Quarry</td>
<td>An area within designated boundaries, approved for the purpose of obtaining rock by sawing or blasting.</td>
</tr>
<tr>
<td>Reconstruction</td>
<td>The process by which a new pavement is constructed, utilizing mostly new materials, to replace an existing pavement.</td>
</tr>
<tr>
<td>Rehabilitation</td>
<td>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.</td>
</tr>
<tr>
<td>Roadbase</td>
<td>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.</td>
</tr>
<tr>
<td>Road Bed</td>
<td>The natural in situ material on which the fill, or in the absence of fill, any pavement layers, are to be constructed.</td>
</tr>
<tr>
<td>Road Bed Material</td>
<td>The material below the subgrade extending to such depth as affects the support of the pavement structure.</td>
</tr>
<tr>
<td>Road Prism</td>
<td>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.</td>
</tr>
<tr>
<td>Roadway</td>
<td>The area normally traveled by vehicles and consisting of one or a number of contiguous traffic lanes, including auxiliary lanes and shoulders.</td>
</tr>
<tr>
<td>Rolling (Terrain)</td>
<td>Terrain with low hills introducing moderate levels of rise and fall with some restrictions on vertical alignment.</td>
</tr>
<tr>
<td>Side Fill</td>
<td>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.</td>
</tr>
<tr>
<td>Side Drain</td>
<td>Open longitudinal drain situated adjacent to and at the bottom of cut or fill slopes.</td>
</tr>
<tr>
<td>Stabilization</td>
<td>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.</td>
</tr>
<tr>
<td>Subbase</td>
<td>The layer of material of specified dimensions on top of the subgrade and below the roadbase. It is the secondary load-spreading layer underlyng 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.</td>
</tr>
<tr>
<td>Subgrade</td>
<td>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.</td>
</tr>
<tr>
<td>Subsurface Drain</td>
<td>Covered drain constructed to intercept and remove subsoil water, including any pipes and permeable material in the drains.</td>
</tr>
<tr>
<td>Surface Treatment</td>
<td>The sealing or resealing of the carriageway or shoulders by means of one or more successive applications of bituminous binder and crushed stone chippings.</td>
</tr>
<tr>
<td>Term</td>
<td>Definition</td>
</tr>
<tr>
<td>-------------------------------</td>
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</tr>
<tr>
<td><strong>Surfacing</strong></td>
<td>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.</td>
</tr>
<tr>
<td><strong>Traffic Lane</strong></td>
<td>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.</td>
</tr>
<tr>
<td><strong>Traffic Volume</strong></td>
<td>Volume of traffic usually expressed in terms of average annual daily traffic (AADT).</td>
</tr>
<tr>
<td><strong>Typical Cross-Section</strong></td>
<td>A cross-section of a road showing standard dimensional details and features of construction.</td>
</tr>
<tr>
<td><strong>Unbound Pavement Materials</strong></td>
<td>Naturally occurring or processed granular material which is not held together by the addition of a binder such as cement, lime or bitumen.</td>
</tr>
<tr>
<td><strong>Wearing Course</strong></td>
<td>The top course of an asphalt surfacing or, for gravel roads, the uppermost layer of construction of the roadway made of specified materials.</td>
</tr>
</tbody>
</table>

**Aggregate**

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

**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.

**Capping Layer**

The top of embankment or bottom of excavation prior to construction of the pavement structure.

**Carriageway**

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

**Construction Joint**

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

**Contraction Joint**

A joint normally placed at recurrent intervals in a rigid slab to control transverse cracking.
Deformed Bar
A reinforcing bar for rigid slabs conforming to “Requirements for Deformations” in AASHTO Designations M 31M.

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

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

Equivalent Standard Axles (ESAS)
Summation of equivalent 8.2 metric ton single axle loads used to combine mixed traffic to design traffic for the design period.

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

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

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

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.

Pavement Layers
The layers of different materials which comprise the pavement structure.

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

Pumping
The ejection of foundation material, either wet or dry, through joints or cracks, or along edges of rigid slabs resulting from vertical movements of the slab under traffic.
<table>
<thead>
<tr>
<th>Term</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcement</td>
<td>Steel embedded in a rigid slab to resist tensile stresses and detrimental opening of cracks.</td>
</tr>
<tr>
<td>Rigid Pavement</td>
<td>A pavement structure which distributes loads to the subgrade, having as one course a Portland cement concrete slab of relatively high-bending resistance.</td>
</tr>
<tr>
<td>Road Bed</td>
<td>The natural in situ material on which the fill, or in the absence of fill, any pavement layers, are to be constructed.</td>
</tr>
<tr>
<td>Road Bed Material</td>
<td>The material below the subgrade extending to such depth as affects the support of the pavement structure.</td>
</tr>
<tr>
<td>Roadway</td>
<td>The area normally traveled by vehicles and consisting of one or a number of contiguous traffic lanes, including auxiliary lanes and shoulders.</td>
</tr>
<tr>
<td>Stabilization</td>
<td>The treatment of the materials used in the construction of the roadbed 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.</td>
</tr>
<tr>
<td>Subbase</td>
<td>The layer of material of specified dimensions on top of the subgrade and, in the case of rigid pavements, below the concrete slab.</td>
</tr>
<tr>
<td>Subgrade</td>
<td>The surface upon which the pavement structure and shoulders are constructed.</td>
</tr>
<tr>
<td>Tie Bar</td>
<td>A deformed steel bar or connector embedded across a joint in a rigid slab to prevent separation of abutting slabs.</td>
</tr>
<tr>
<td>Traffic Lane</td>
<td>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.</td>
</tr>
<tr>
<td>Welded Wire Fabric</td>
<td>Welded Steel Wire Fabric for Concrete Reinforcement.</td>
</tr>
</tbody>
</table>
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
BCEOM Bureau Central d’Etudes d’Outre Mer (France)
BS    British Standard
CBR   California Bearing Ratio (as described in AASHTO T 193)
CEBTP Centre d’Etudes du Batiment et des Travaux Publics (France)
CRCP  Continuously Reinforced Concrete Pavement
CSRA  Committee of State Road Authorities (South Africa)
DCP   Dynamic Cone Penetrometer

Drainage Coefficients (m2, m3)
Factors used to modify layer coefficients in flexible pavements as a function of how well the pavement structure can handle the adverse effect of water infiltration

ERA   Ethiopian Roads Agency
ESA   Equivalent Single Axle

Equivalent Standard Axles (ESA)
Summation of equivalent 8.2 t single axle loads used to combine mixed traffic to design traffic class

FWD   Falling Weight Deflectometer
IRI   International Roughness Index
ICL   Initial Consumption of Lime Test
Is    Global damage index (used in the VIZIR method)
JPCP  Jointed Plain Concrete Pavement
JRCP  Jointed Reinforced Concrete Pavement

Layer Coefficients (a1, a2, a3)
The empirical relationship between structural number (SN) and layer thickness which expresses the relative ability of a material to function as a structural component of the pavement

LCPC Laboratoire Central des Ponts et Chaussees (France)
<table>
<thead>
<tr>
<th>Term</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maintenance</td>
<td>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.</td>
</tr>
<tr>
<td>MDD</td>
<td>Maximum Dry Density</td>
</tr>
<tr>
<td>NDT</td>
<td>Non-Destructive Testing</td>
</tr>
<tr>
<td>Overlay</td>
<td>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.</td>
</tr>
<tr>
<td>PCC</td>
<td>Portland Cement Concrete</td>
</tr>
<tr>
<td>PMS</td>
<td>Pavement Management System</td>
</tr>
<tr>
<td>Q&lt;sub&gt;i&lt;/sub&gt;</td>
<td>Pavement Quality Rating (used in the VIZIR method)</td>
</tr>
<tr>
<td>Reconstruction</td>
<td>The process by which a new pavement is constructed, utilizing mostly new materials, to replace an existing pavement.</td>
</tr>
<tr>
<td>Recycling</td>
<td>The reuse, usually after some processing, of a material that has already served its first-intended purpose.</td>
</tr>
<tr>
<td>Rehabilitation</td>
<td>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.</td>
</tr>
<tr>
<td>Roadbase</td>
<td>A layer of material of defined thickness and width constructed on top of the subbase, or in the absence thereof, the subgrade.</td>
</tr>
<tr>
<td>RRD</td>
<td>Representative Rebound Deflection</td>
</tr>
<tr>
<td>S1 to S6</td>
<td>Subgrade strength classes used to characterize the subgrade in pavement design</td>
</tr>
<tr>
<td>Structural Number (SN)</td>
<td>An index number into which thickness of flexible pavement layers may be converted through the use of suitable layer coefficients related to the type of material being used in each layer of the pavement structure</td>
</tr>
<tr>
<td>SN&lt;sub&gt;eff&lt;/sub&gt;</td>
<td>Effective structural number of an existing pavement.</td>
</tr>
<tr>
<td>SN&lt;sub&gt;new&lt;/sub&gt;</td>
<td>Structural number of a new pavement.</td>
</tr>
<tr>
<td>Subbase</td>
<td>The layer of material of specified dimensions on top of the subgrade and below the roadbase.</td>
</tr>
<tr>
<td>Subgrade</td>
<td>The surface upon which the pavement structure and shoulders are constructed.</td>
</tr>
<tr>
<td>Surfacing</td>
<td>The asphalt surfacing of a flexible pavement or the concrete slab of a rigid pavement.</td>
</tr>
<tr>
<td>T1 to T8</td>
<td>Traffic classes used to characterize the anticipated traffic in terms of ESA for flexible pavement design purposes</td>
</tr>
<tr>
<td>T&lt;sub&gt;1&lt;/sub&gt;, T&lt;sub&gt;2&lt;/sub&gt;, T&lt;sub&gt;3&lt;/sub&gt;</td>
<td>Thicknesses (in centimeters) of pavement surface, base and subbase layers (existing or required)</td>
</tr>
<tr>
<td>TRL</td>
<td>Transport Research Laboratory (UK)</td>
</tr>
<tr>
<td>Acronym</td>
<td>Description</td>
</tr>
<tr>
<td>---------</td>
<td>-------------</td>
</tr>
<tr>
<td>VIZIR</td>
<td>A method for quality evaluation of paved roads developed by the LCPC and used in PMS implementation</td>
</tr>
<tr>
<td>VOC</td>
<td>Vehicle Operating Cost</td>
</tr>
<tr>
<td>VMA</td>
<td>Voids in mineral aggregate</td>
</tr>
</tbody>
</table>
REFERENCES


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