TRANSPORT RESEARCH LABORATORY

ODA

OVERSEAS ROAD NOTE 31

A guide to the structural design of bitumen-surfaced roads in tropical and sub-tropical countries

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OVERSEAS ROAD NOTE 31

(FOURTH EDITION)

A GUIDE TO THE STRUCTURAL DESIGN OF BITUMEN-SURFACED ROADS IN TROPICAL AND SUB-TROPICAL COUNTRIES

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OVERSEAS ROAD NOTES

Overseas Road Notes are prepared principally for road and transport authorities in countries receiving technical assistance from the British Government. A limited number of copies is available to other organisations and to individuals with an interest in roads overseas. Enquiries should be made to:

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FOREWORD

Road Note 31 was first published in 1962 and revised in 1966 and 1977 to take account of advances in our understanding of the behaviour of road-budding materials and their interaction in composite pavements. Many of these advances have been made by engineers and scientists working in temperate climates but a considerable amount of both fundamental and applied research has been necessary to adapt and develop the knowledge so that it can be used with confidence in tropical and sub-tropical regions where conditions are often very different. In addition to differences associated with climate and types of materials, problems also arise in some countries from uncontrolled vehicle loading and unreliable road maintenance. At the same time, the level of technology available for construction and maintenance can be relatively low.

All this has presented a unique challenge to the highway engineer. This edition of the Road Note has drawn on the experience of TRL and collaborating organisations in over 30 tropical and sub-tropical countries. Most of this experience has been gained in carrying out a research and development programme on behalf of the Overseas Development Administration, with additional projects for other aid agencies. The new edition extends the designs of previous editions to cater for traffic up to 30 million equivalent standard axles and takes account of the variability in material properties and construction control, the uncertainty in traffic forecasts, the effects of climate and high axle loads and the overall statistical variability in road performance. The range of structures has been expanded and the chapters on the different types of materials have been enlarged to provide more detailed advice on specifications and techniques. Nevertheless, there will be situations and conditions which are not covered in the Note and there will be many examples where local knowledge can be used to refine and improve the recommendations. Indeed, the role of local engineering knowledge and the judgement of experienced engineers should never be underestimated and should always form an important part of the design process.

The Note has been produced in response to a continuous demand from engineers worldwide and I am confident that the new edition will prove to be as popular with practitioners as its predecessors.

John Hodges
Chief Engineering Adviser
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CONTENTS

A GUIDE TO THE STRUCTURAL DESIGN OF BITUMEN-SURFACED ROADS IN TROPICAL AND SUB-TROPICAL COUNTRIES

1. INTRODUCTION

1.1 GENERAL

This Road Note gives recommendations for the structural design of bituminous surfaced roads in tropical and subtropical climates It is aimed at highway engineers responsible for the design and construction 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. The design of strengthening overlays is not covered nor is the design of earth, gravel or concrete roads. Although this Note is appropriate for the structural design of flexible roads in urban areas, some of the special requirements of urban roads, such as the consideration of kerbing, sub-soil drainage, skid resistance, etc , are not covered.

For the structural design of more heavily trafficked roads, the recommendations of this Note may be supplemented by those given in the guides for the design of bituminous pavements in the. United Kingdom (Powell et al (1984)) but these are likely to require some form of calibration or adaptation to take account of the conditions encountered in the tropics.

1.2 ROAD DETERIORATION

The purpose of structural design is to limit the stresses induced in the subgrade by traffic to a safe level at which subgrade deformation is insignificant whilst at the same time ensuring that the road pavement layers themselves do not deteriorate to any serious extent within a specified period of time.

By the nature of the materials used for construction, it is impossible to design a road pavement which does not deteriorate in some way with time and traffic, hence the aim of structural design is to limit the level of pavement distress, measured primarily in terms of riding quality, rut depth and cracking, to predetermined values. Generally these values are set so that a suitable remedial treatment at the end of the design period is a strengthening overlay of some kind but this is not necessarily so and roads can, in principle, be designed to reach a terminal condition at which mayor rehabilitation or even complete reconstruction is necessary. However, assessing appropriate remedial treatments for roads which have deteriorated beyond a certain level is a difficult task. In most design methods it is assumed that adequate routine and periodic maintenance is carved out during the design period of the road and that at the end of the design period a relatively low level of deterioration has occurred

Acceptable levels of surface condition have usually been based on the expectations of road users. These expectations have been found to depend upon the class of road and the volume of traffic such that the higher the geometric standard, and therefore the higher the vehicle speeds, the lower the level of pavement distress which is acceptable. In defining these levels, economic considerations were not considered because there was insufficient knowledge of the cost trade-offs for an economic analysis to be carved out with sufficient accuracy.

1.3 ECONOMIC CONSIDERATIONS

In recent years a number of important empirical studies have shown how the costs of operating vehicles depend on the surface condition of the road. The studies have also improved our knowledge of how the deterioration of roads depends on the nature of the traffic, the properties of the road-making materials, the environment, and the maintenance strategy adopted (Parsley and Robinson (1982), Paterson (1987), Chesher and Harrison (1987), Watanatada et al (1987)). In some circumstances it is now possible 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 transport facility i e the sum of construction costs, maintenance costs and road user costs, can be minimised. These techniques are expected to become more widespread in the future. Also, with the introduction in many countries of pavement management systems in which road condition is monitored on a regular basis, additional information will be collected to allow road performance models to be refined. Pavement structural design could 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. Whilst these refinements lie in the future, the research has provided important guidance on structural designs suitable for tropical and sub-tropical environments and has been used, in part, in preparing this edition of Road Note 31.

For the structures recommended in this Note, the level of deterioration that is reached by the end of the design period has been restricted to levels that experience has shown give rise to acceptable economic designs under a wide range of conditions. It has been assumed that routine and periodic maintenance activities are carried out to a reasonable, though not excessive, level In particular, it has been assumed that periodic maintenance is done whenever the area of road surface experiencing defects i e cracking, ravelling, etc , exceeds 15 per cent. For example, for a 10 year design period, one surface maintenance treatment is likely to be required for the higher traffic levels whereas for a 15 year design period, one treatment is likely to be required for the lower traffic levels and two for the higher. These are broad guidelines only and the exact requirements will depend on local conditions.

1.4 EFFECTS OF CLIMATE

Research has shown how different types of road deteriorate and has demonstrated that some of the most common modes of failure in the tropics are often different from those encountered in temperate regions. In particular, climate related deterioration sometimes dominates performance and the research has emphasised the overriding importance of the design of bituminous surfacing materials to minimise this type of deterioration (Paterson (1987), Smith et al (1990), Strauss et al (1984)). This topic is dealt with in Chapter 8.

Climate also affects the nature of the soils and rocks encountered in the tropics Soil-forming processes are still very active and the surface rocks are often deeply weathered. The soils themselves often display extreme or unusual properties which can pose considerable problems for road designers. The recent publication 'Road building in the tropics materials and methods' provides an introduction to these topics (Millard (1993)).

1.5 VARIABILITY IN MATERIAL PROPERTIES AND ROAD PERFORMANCE

Variability in material properties and construction control is generally much greater than desired by the design engineer and must be taken into account explicitly in the design process. Only a very small percentage of the area of the surface of a road needs to show distress for the road to be considered unacceptable by road users. It is therefore the weakest parts of the road or the extreme tail of the statistical distribution of 'strength' which is important in design. In well controlled full-scale experiments this variability is such that the ten per cent of the road which performs best will carry about six times more traffic before reaching a defined terminal condition than the ten per cent which performs least well. Under normal construction conditions this spread of performance becomes even greater. Some of this variability can be explained through the measured variability of those factors known to affect performance. Therefore, if the likely variability is known beforehand, it is possible, in principle, for it to be taken into account in design. It is false economy to minimise the extent of preliminary. investigations to determine this variability.

In practice it is usually only the variability of subgrade strength that is considered and all other factors are controlled by means of specifications i e by setting minimum acceptable values for the key properties. But specifications need to be based on easily measurable attributes of the materials and these may not correlate well with the fundamental mechanical properties on which behaviour depends. As a result, even when the variability of subgrade strength and pavement material properties. are taken into account, there often remains a considerable variation in performance between nominally identical pavements which cannot be fully explained. 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. Thus there is always likely to be scope for improving designs based on local experience.

Nevertheless, it is the task of the designer to estimate likely variations in layer thicknesses and material strengths so that realistic target values and tolerances can be set in the specifications to ensure that satisfactory road performance can be guaranteed as far as is possible.

The thickness and strength values described in this Road Note are essentially minimum values but practical considerations require that they are interpreted as lower ten percentile values with 90 per cent of all test results exceeding the values quoted. The random nature of variations in thickness and strength which occur when each layer is constructed should ensure that minor deficiencies in thickness or strength do not occur one on top of the other, or very rarely so. The importance of good practice in quarrying, material handling and stock-piling to ensure this randomness and also to minimise variations themselves cannot be over emphasised.

1.6 UNCERTAINTY IN TRAFFIC FORECASTS

Pavement design also depends on the expected level of traffic. Axle load studies and traffic counts are essential prerequisites for successful design but traffic forecasting remains a difficult task and therefore sensitivity and risk analysis are recommended. This topic is discussed in Chapter 2.

1.7 BASIS FOR THE DESIGN CATALOGUE

The pavement designs incorporated into the fourth edition of Road Note 31 are based primarily on:

(a) The results of full-scale experiments where all factors affecting performance have been accurately measured and their variability quantified.

(b) Studies of the performance of as-built existing road networks.

Where direct empirical evidence is lacking, designs have been interpolated or extrapolated from empirical studies using road performance models (Parsley and Robinson (1982), Paterson (1987), Rolt et al (1987)) and standard analytical, mechanistic methods e.g Gerritsen and Koole (1987), Powell et al (1984), Brunton et al (1987).

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 behaviour, the design charts have been presented as a catalogue of structures, each structure being applicable over a small range of traffic and subgrade strength. Such a procedure makes the charts extremely easy to use but it is important that the reader is thoroughly conversant with the notes applicable to each chart

Throughout the text the component layers of a flexible pavement are referred to in the following terms (see Figure 1).

Surfacing. This is the uppermost layer of the pavement and will normally consist of a bituminous surface dressing or a layer of premixed bituminous material. Where premixed materials are laid in two layers, these are known as the wearing course and the basecourse (or binder course) as shown in Figure 1.

Roadbase. This is the main load-spreading layer of the pavement. It will normally consist of crushed stone or gravel, or of gravelly soils, decomposed rock, sands and sand-clays stabilised with cement, lime or bitumen.

Sub-base. This is the secondary load-spreading layer underlying the roadbase. It will normally consist of a material of lower quality than that used in the roadbase such as unprocessed natural gravel, gravel-sand, or gravel-sand-clay. This layer also serves as a separating layer preventing contamination of the roadbase by the subgrade material and, under wet conditions, it has an important role to play in protecting the subgrade from damage by construction traffic.

Capping layer (selected or improved subgrade). Where very weak soils are encountered, a capping layer is sometimes necessary. This may consist of better quality subgrade material imported from elsewhere or existing subgrade material improved by mechanical or chemical stabilisation.

Subgrade. This is the upper layer of the natural soil which may be undisturbed local material or may be soil excavated elsewhere and placed as fill. In either case it is compacted during construction to give added strength.

1.8 THE DESIGN PROCESS

There are three main steps to be followed in designing a new road pavement These are:

(i) estimating the amount of traffic and the cumulative number of equivalent standard axles that will use the road over the selected design life;

(ii) assessing the strength of the subgrade soil over which the road is to be built;

(iii) selecting the most economical combination of pavement materials and layer thicknesses that will provide satisfactory service over the design life of the pavement (It is usually necessary to assume that an appropriate level of maintenance is also carried out).

This Note considers each of these steps in turn and puts special emphasis on five aspects of design that are of major significance in designing roads in most tropical countries:

- The influence of tropical climates on moisture conditions in road subgrades.
- The severe conditions imposed on exposed bituminous surfacing materials by tropical climates and the implications of this for the design of such surfacings.
- The interrelationship between design and maintenance. If an appropriate level of maintenance cannot be assumed. it is not possible to produce designs that will carry the anticipated traffic loading without high costs to vehicle operators through increased road deterioration.
- The high axle loads and tyre pressures which are common in most countries.
- The influence of tropical climates on the nature of the soils and rocks used in road building.

The overall process of designing a road is illustrated in Figure 2. Some of the information necessary to carry out the tasks may be available from elsewhere e.g a feasibility study or Ministry records, but all existing data will need to be checked carefully to ensure that it is both up-to-date and accurate. Likely problem areas are highlighted in the relevant chapters of this Note.

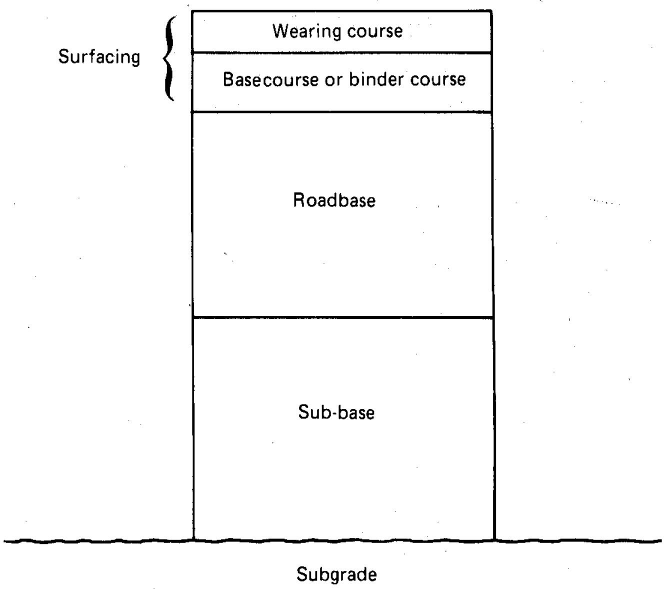


Fig. 1 Definition of pavement layers

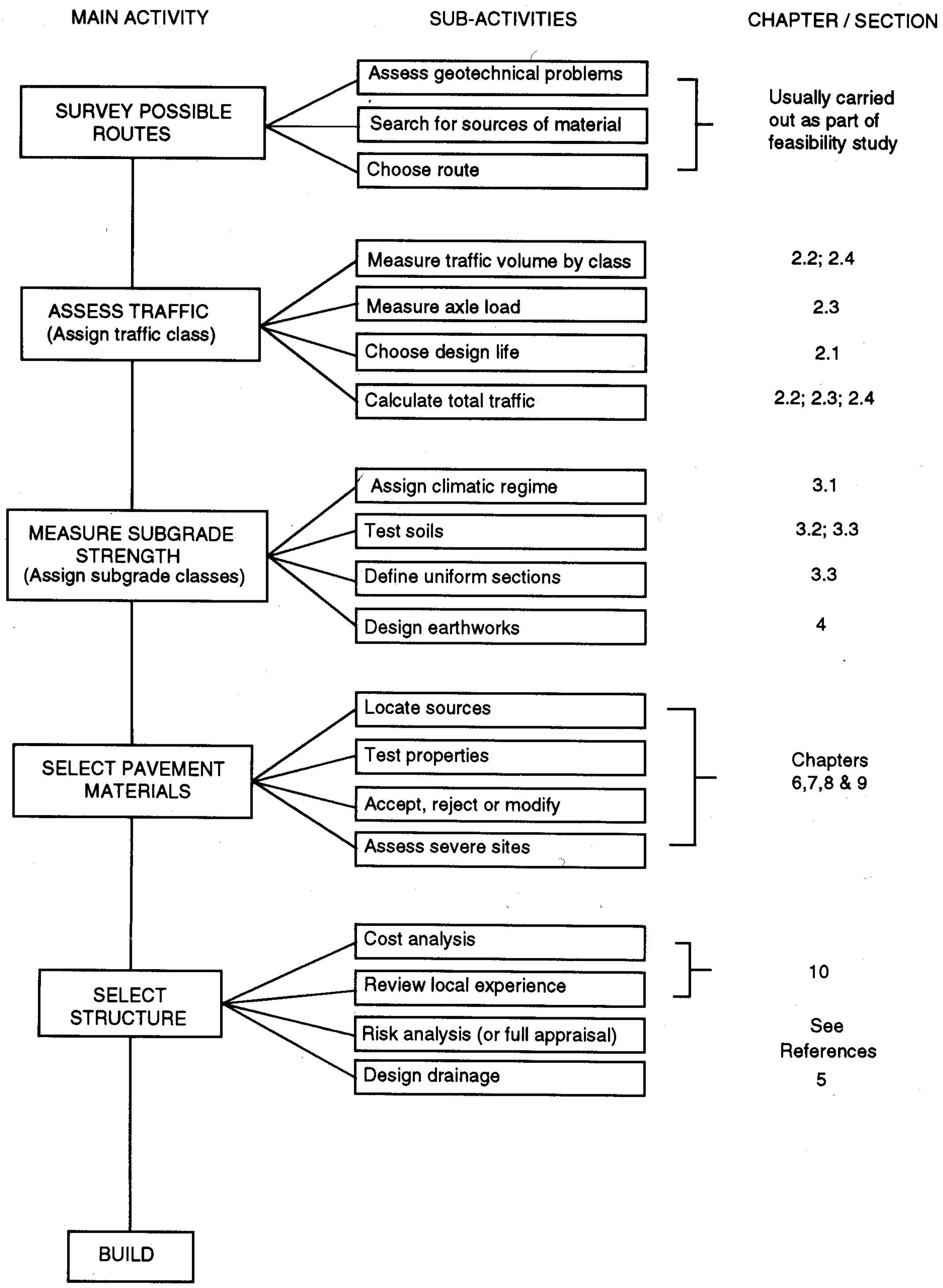


Fig. 2 The pavement design process

2. TRAFFIC

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. For pavement design purposes 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. The loads imposed by private cars do not contribute significantly to the structural damage. For the purposes of structural design, cars and similar sized vehicles can be ignored and only the total number and the axle loading of the heavy vehicles that will use the road during its design life need to be considered In this context, heavy vehicles are defined as those having an unladen weight of 3000 kg or more. In some circumstances, particularly for lightly trafficked roads, construction traffic can be a significant component of overall traffic loading and the designs should take this into account.

2.1 DESIGN LIFE

For most road projects an economic analysis period of between 10 and 20 years from the date of opening is appropriate, but for major projects this period should be tested as part of the appraisal process (Overseas Road Note 5, Transport and Road Research Laboratory (1988)). Whatever time period is chosen for the appraisal of a project, the road will always have some residual value at the end of this period. Choosing a pavement design life that is the same as the analysis period simplifies the economic appraisal by minimising the residual value, which is normally difficult to estimate accurately. A pavement design life of 15 years also reduces the problem of forecasting uncertain traffic trends for long periods into the future.

In this context, design life does not mean that at the end of the period the pavement will be completely worn out and in need of reconstruction; it means that towards the end of the period the pavement will need to be strengthened so that it can continue to carry traffic satisfactorily for a further period. Condition surveys of bituminous pavements should be carried out about once a year as part of the inspection procedures for maintenance. These are used to determine not only the maintenance requirements but also the nature and rate of change of condition to help to identify if and when the pavement is likely to need strengthening.

Stage construction consists of planned improvements to the pavement structure at fixed times through the project life. From a purely economic point of view, stage construction policies have much to commend them. However, experience has shown that budget constraints have often prevented the planned upgrading phases of stage construction projects from taking place, with the result that much of the benefit from such projects has been lost in general, stage construction policies are not recommended if there is any risk that maintenance and upgrading will not be carved out correctly or at the appropriate time.

2.2 ESTIMATING TRAFFIC FLOWS

2.2.1 Baseline traffic flows

In order to determine the total traffic over the design life of the road, the first step is to estimate baseline traffic flows. The estimate should be the (Annual) Average Daily Traffic (ADT) currently using the route, classified into the vehicle categories of cars, light goods vehicles, trucks (heavy goods vehicles) and buses. The ADT is defined as the total annual traffic summed for ***both*** directions and divided by 365. It is usually obtained by recording actual traffic flows over a shorter period from which the ADT is then estimated. For long projects, large differences in traffic along the road may make it necessary to estimate the flow at several locations.. It should be noted that for structural design purposes the traffic loading in ***one*** direction is required and for this reason care is always required when interpreting ADT figures.

Traffic counts carried out over a short period as a basis for estimating the traffic flow can produce estimates which are subject to large errors because traffic flows can have large daily, weekly, monthly and seasonal variations (Howe (1972)). The daily variability in traffic flow depends on the volume of traffic. It increases as traffic levels fall, with high variability on roads carrying less than 1000 vehicles per day. Traffic flows vary more from day-to-day than from week-to-week over the year. Thus there are large errors associated with estimating average daily traffic flows (and subsequently annual traffic flows) 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 flows also vary from month-to-month 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 duration Traffic also varies considerably through a 24-hour period and this needs to be taken into account explicitly as outlined below.

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

(i) The counts are for seven consecutive days.

(ii) The counts on some of the days are for a full 24 hours, with preferably 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 grossed up 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 re-peated several times throughout the year.

Country-wide traffic data should be collected on a systematic basis to enable seasonal trends in traffic flows to be quantified. Unfortunately, many of the counts that are available are unreliable, especially if they have been carried out manually. Therefore, where seasonal adjustment factors are applied to traffic survey data in order to improve the accuracy of baseline traffic figures, the quality of the statistics on which they are based should be checked in the field.

Classified traffic counts are normally obtained by counting manually. These counts can be supplemented by automatic counters which use either a pneumatic tube laid across the surface of the carriageway or a wire loop fixed to the carriageway surface or, preferably, buried just beneath it. Pneumatic tubes require regular maintenance and can be subject to vandalism. Buried loops are preferred, but installing a loop beneath the road surface is more difficult and more expensive than using a pneumatic tube.

In their basic form, automatic counters do not distinguish between different types of vehicle and so cannot provide a classified count. Modern detector systems are now available which can perform classified vehicle counting, but such systems are expensive and not yet considered to be sufficiently robust for most developing country applications. An exception is the counter system developed specifically for developing countries by the Transport Research Laboratory.

2.2.2 Traffic forecasting

Even with a developed economy and stable economic conditions, traffic forecasting is an uncertain process. In a developing economy the problem becomes more difficult because such economies are often very sensitive to the world prices of just one or two commodities.

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 commonest method of forecasting normal traffic is to extrapolate time series data on traffic levels and assume that growth will either remain constant in absolute terms i.e. a fixed number of vehicles per year (a linear extrapolation), or constant in relative terms i.e. a fixed percentage increase. Data on fuel sales can often be used as a guide to country-wide growth in traffic levels, although improvements in fuel economy over time should be taken into account. 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, but it has the disadvantage that a forecast of GDP is needed. The use of additional variables, such as population or fuel price, brings with it the same problem If GDP forecasts are not available, then future traffic growth should be based on time series data.

If it is thought that a particular component of the traffic will grow at a different rate to the rest, it should be specifically identified and dealt with separately. For example, there may be a plan to expand a local town or open a local factory during the design life of the road, either of which could lead to different growth rates for different types of vehicle, or there may be a plan to allow larger freight vehicles on the road, in which case the growth rate for trucks may be relatively low because each truck is heavier.

Whatever the forecasting procedure used, it is essential to consider the realism of forecast future levels. Few developing countries are likely to sustain the high rates of growth experienced in the past, even in the short term, and factors such as higher fuel costs and vehicle import restrictions could tend to depress future growth rates.

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 be carried out to provide data on the traffic diversions likely to arise. Assignment of diverted traffic is normally done by an all-or-nothing method in which it is assumed that all vehicles that would save time or money by diverting would do so, and that vehicles that would lose time or increase costs would not transfer. With such a method it is important that all perceived costs are included. In some of the more developed countries there may be scope for modelling different scenarios using standard assignment computer programs.

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. However, quality of service, speed and convenience are valued by intending consignors and, for general goods, diversion from other modes should not be estimated solely on the basis of door-to-door transport charges. Similarly, the choice of mode for passenger transport should not be judged purely on the basis of travel charges. The importance attached to quality of service by users has been a major contributory factor to the worldwide decline in rail transport over recent years.

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 difficult to forecast accurately and can be easily overestimated It is only likely to be significant in those cases where the road investment brings about large reductions in transport costs. For example, in the case of a small improvement within an already developed highway system, generated traffic will be small and can normally be ignored. However, in the case of a new road allowing access to a hitherto undeveloped area, there could be large reductions in transport costs as a result of changing mode from, for example, animal-based transport to motor vehicle transport. In such a case, generated traffic could be the main component of future traffic flow.

The recommended approach to forecasting generated traffic is to use demand relationships. The price elasticity of demand for transport is the responsiveness of traffic to a change in transport costs following a road investment. On inter-urban roads a distinction is normally drawn between passenger and freight traffic. On roads providing access to rural areas, a further distinction is usually made between agricultural and non-agricultural freight traffic.

Evidence from several evaluation studies carried out in developing countries gives a range of between -0 6 to -2.0 for the price elasticity of demand for transport, with an average of about -1 0. This means that a one per cent decrease in transport costs leads to a one per cent increase in traffic. Calculations should be based on door-to-door travel costs estimated as a result of origin and destination surveys and not lust on that part of the trip incurred on the road under study.

The available evidence suggests that the elasticity of demand for passenger travel is usually slightly greater than unity In general, the elasticity of demand for goods is much lower and depends on the proportion of transport costs in the commodity price.

2.3 AXLE LOADING

2.3.1 Axle equivalency

The damage that vehicles do to a road depends very strongly 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 tonnes using equivalence factors which have been derived from empirical studies (Highway Research Board (1962), Paterson (1987)). 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 (esa).

Axle load surveys must be carried out to determine the axle load distribution of a sample of the heavy vehicles using the road. Data collected from these surveys are used to calculate the mean number of equivalent standard axles 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.

2.3.2 Axle load surveys

If no recent axle load data are available it is recommended that axle load surveys of heavy vehicles are undertaken whenever a major road project is being designed. Ideally, several surveys at periods which will reflect seasonal changes in the magnitude of axle loads are recommended. Portable vehicle-wheel weighing devices are available which enable a small team to weigh up to 90 vehicles per hour. Detailed guidance on carrying out axle load surveys and analysing the results is given in TRRL Road Note 40 (Transport and Road Research Laboratory (1978)).

It is recommended that axle load surveys are carried out by weighing a sample of vehicles at the roadside. The sample should be chosen such that a maximum of about 60 vehicles per hour are weighed. The weighing site should be level and, if possible, constructed in such a way that vehicles are pulled clear of the road when being weighed. The portable weighbridge should be mounted in a small pit with its surface ***level*** with the surrounding area. This ensures that all of the wheels of the vehicle being weighed are level and eliminates the errors which can be introduced by even a small twist or tilt of the vehicle. More importantly, it also eliminates the large errors that can occur if all the wheels on one side of multiple axle groups are not kept in the same horizontal plane. The load distribution between axles in multiple axle groups is often uneven and therefore each axle must be weighed separately. The duration of the survey should be based on the same considerations as for traffic counting outlined in Section 2 2 1.

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

2.3.3 Determination of cumulative equivalent standard axles

Computer programs have been written to assist with the analysis of the results from axle load surveys. These programs provide a detailed tabulation of the survey results and determine the mean equivalence factors for each vehicle type if required. If such a program is not available, standard spreadsheet programs can be used.

If there are no computer facilities available the following method of analysis is recommended. The equivalence factors for each of the wheel loads measured during the axle load survey are determined using Table 2.1 or the accompanying equation to obtain the equivalence factors for vehicle axles. The factors for the axles are totalled to give the equivalence 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.

The mean equivalence factor for each type or class of vehicle travelling in each direction must then be determined. Vehicle classes are usually defined by the number and type of axles. Note that this method of determining the mean equivalence factors must always be used; calculating the equivalence factor for the average axle load is incorrect and leads to large errors.

In order to determine the cumulative equivalent standard axles over the design life of the road, the following procedure should be followed:

(i) Determine the daily traffic flow for each class of vehicle weighed using the results of the traffic survey and any other recent traffic count information that is available.

(ii) Determine the average daily one-directional traffic flow for each class of vehicle.

(iii) Make a forecast of the one-directional traffic flow for each class of vehicle to determine the total traffic in each class that will travel over each lane during the design life (see Section 2.2.2).

(iv) Determine the mean equivalence factor of each class of vehicle and for each direction from the results of this axle load survey and any other surveys that have recently been carried out.

(v) The products of the cumulative one-directional traffic flows for each class of vehicle over the design life of the road and the mean equivalence factor for that class should then be calculated and added together to give the cumulative equivalent standard axle loading for each direction. The higher of the two directional values should be used for design.

TABLE 2.1

Equivalence factors for different axle loads

Wheel load (single & dual) (10^3 kg)	Axle load (10^3 kg)	Equivalence factor
1.5	3.0	0.01
2.0	4.0	0.04
2.5	5.0	0.11
3.0	6.0	0.25
3.5	7.0	0.50
4.0	8.0	0.91
4.5	9.0	1.55
5.0	10.0	2.50
5.5	11.0	3.83
6.0	12.0	5.67
6.5	13.0	8.13
7.0	14.0	11.3
7.5	15.0	15.5
8.0	16.0	20.7
8.5	17.0	27.2
9.0	18.0	35.2
9.5	19.0	44.9
10.0	20.0	56.5

$$\text{Equivalence factor} = \left(\frac{\text{Axle load (kg)}}{8160}\right)^{4.5}$$

In most countries the axle load distribution of the total population of heavy vehicles using the road system remains roughly constant from year to year although there may be long-term trends resulting from the introduction of new types of vehicles or changes in vehicle regulations and their enforcement. It is therefore customary to assume that the axle load distribution of the heavy vehicles will remain unchanged for the design life of the pavement and that it can be determined by undertaking surveys of vehicle axle loads on existing roads of the same type and which serve the same function. In most developing countries the probable errors in these assumptions for a design life of 15 years are unlikely to result in a significant error in design.

On dual carriageway roads and on single carriageway roads with more than two lanes, it should be assumed that the slow traffic lanes will carry all the heavy vehicles unless local experience indicates otherwise or the traffic flow exceeds about 2000 heavy vehicles per day in each direction. In the latter case, a proportion of heavy vehicles should be assigned to the slow lane according to the principles outlined in Overseas Road Note No. 6 (Transport and Road Research Laboratory (1988)). The design thickness required for the slow lane is usually applied to the whole carriageway width but there may be situations where a tapered roadbase or sub-base is appropriate.

In some countries, single-lane bituminous roads are built to economise on construction costs. On such roads the traffic tends to be more channelised than on two-lane roads. The effective traffic loading in the wheelpath in one direction has been shown to be ***twice*** that for a wider road. Therefore, taking into account the traffic in both directions, the pavement thickness for these roads should be based on ***four*** times the total number of heavy vehicles that travel in one direction.

2.4 ACCURACY

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 minimise these errors there is no substitute for carrying out specific traffic surveys for each project for the durations suggested in Section 2.2.1. 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 relatively insensitive to cumulative axle load and the method recommended in this. Note provides fixed structures for ranges of traffic as shown in Table 2.2. As long as the estimate of cumulative equivalent standard axles is close to the centre 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. Formal risk analysis can also be used to evaluate the design choices as described briefly and referenced in Overseas Road Note 5 (Transport and Road Research Laboratory (1988)).

TABLE 2.2

Traffic classes

Traffic classes	Range (10^6 esa)
T1	< 0.3
T2	0.3 - 0.7
T3	0.7 - 1.5
T4	1.5 - 3.0
T5	3.0 - 6.0
T6	6.0 - 10
T7	10 - 17
T8	17 - 30

3. THE SUBGRADE

The type of subgrade sod is largely determined by the location of the road, but 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.

The strength of road subgrades 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.

For designing the thickness of a road pavement, the strength of the subgrade should be taken as that of the soil at a moisture content equal to the wettest moisture condition likely to occur in the subgrade after the road is opened to traffic. In the tropics, subgrade moisture conditions under ***impermeable*** road pavements can be classified into three main categories:

Category (1). Subgrades where the water table is sufficiently close to the ground surface to control the subgrade moisture content.

The type of subgrade soil governs the depth below the road surface at which a water table becomes the dominant influence on the subgrade moisture content. 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 per cent) the water table will dominate when it rises to within 3m of the road surface, and in heavy clays (PI>40 per cent) the water table will dominate when it rises to within 7m of the road surface.

In addition to areas where the water table is maintained by rainfall, this category includes coastal strips and flood plains where the water table is maintained by the sea, by a lake or by a river.

Category (2). Subgrades with deep water tables and where rainfall is sufficient to produce significant changes in moisture conditions under the road.

These conditions occur when rainfall exceeds evapotranspiration for at least two months of the year. The rainfall in such areas is usually greater than 250 mm per year and is often seasonal.

Category (3). Subgrades in areas with no permanent water table near the ground surface and where the climate is dry throughout most of the year with an annual rainfall of 250 mm or less.

Direct assessment of the likely strength or CBR of the subgrade soil is often difficult to make but its value can be inferred from an estimate of the density and equilibrium (or ultimate) moisture content of the subgrade together with knowledge of the relationship between strength, density and moisture content for the soil in question. This relationship must be determined in the Laboratory. The density of the subgrade soil can be controlled within limits by compaction at a suitable moisture content at the time of construction. The moisture content of the subgrade soil is governed by the local climate and the depth of the water table below the road surface. In most circumstances, the first task is therefore to estimate the equilibrium moisture content as outlined in Section 3 1 below. A method of direct assessment of the subgrade strength, where this is possible, is discussed in Section 3 2 together with less precise methods of estimation which can be used if facilities for carrying out the full procedure are not available.

3.1 ESTIMATING THE SUBGRADE MOISTURE CONTENT

Category (7). The easiest method of estimating the design subgrade moisture content is to measure the moisture content in subgrades below existing pavements in similar situations at the time of the year 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. If there is no suitable road in the vicinity, the moisture content in the subgrade under an impermeable pavement can be estimated from a knowledge of the depth of the water table and the relationship between suction and moisture content for the subgrade soil (Russam and Croney (1960)). The test apparatus required for determining this relationship is straightforward and the method is described in Appendix B.

Category (2). When the water table is not near the ground surface, the subgrade moisture condition under an impermeable pavement will depend on the balance between the water entering the subgrade through the shoulders and at the edges of the pavement during wet weather and the moisture leaving the ground by evapotranspiration during dry periods. Where the average annual rainfall is greater than 250 mm a year, the moisture condition for design purposes can be taken as the optimum moisture content given by the British Standard (Light) Compaction Test, 2.5 kg rammer method.

When deciding on the depth of the water table in Category (1) or Category (2) subgrades, the possibility of the existence of local perched water tables should be borne in mind and the effects of seasonal flooding (where this occurs) should not be overlooked.

Category (3). In regions where the climate is dry throughout most of the year (annual rainfall 250 mm or less), the moisture content of the subgrade under an impermeable pavement will be low. For design purposes a value of 80 per cent of the optimum moisture content obtained in the British Standard (Light) Compaction Test, 2.5 kg rammer method, should be used.

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, stabilised 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} metres 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 Category (1) and Category (2) should be assessed on the basis of saturated CBR samples as described in Section 3.2. 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 the British Standard (Light) Compaction Test, 2 5 kg rammer method.

3.2 DETERMINING THE SUBGRADE STRENGTH

Having estimated the subgrade moisture content for design, it is then possible to determine the appropriate design CBR value at the specified density. It is recommended that the top 250 mm of all subgrades should be compacted during construction to a relative density of at least 100 per cent of the maximum dry density achieved in the British Standard (Light) Compaction Test, 2 5 kg rammer method, or at least 93 per cent of the maximum dry density achieved in the British Standard (Heavy) Compaction Test using the 4.5 kg rammer. With modern compaction plant a relative density of 95 per cent of the density obtained in the heavier compaction test should be achieved without difficulty but tighter control of the moisture content will be necessary. Compaction will not only improve the subgrade bearing strength but will reduce permeability and subsequent compaction by traffic.

As a first step it is necessary to determine the compaction properties of the subgrade soil by carrying out standard laboratory compaction tests. Samples of the subgrade soil at the design subgrade moisture content can then be compacted in CBR moulds to the specified density and tested to determine the CBR values.

With cohesionless sands, the rammer method tends to overestimate the optimum moisture content and underestimate the dry density achieved by normal field equipment. The vibrating hammer method is more appropriate for these materials.

If samples of cohesive soils are compacted at moisture contents equal to or greater than the optimum moisture content, they should be left sealed for 24 hours before being tested so that excess pore water pressures induced during compaction are dissipated.

Alternatively, a more complete picture of the relationship between density, moisture content and CBR for the subgrade soil can be obtained by measuring the CBR of samples compacted at several moisture contents and at least two levels of compaction. The design CBR is then obtained by interpolation. This method is preferable since it 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 calculated. Figure 3 shows a typical dry density/moisture content/CBR relationship for a sandy-clay soil that was obtained by compacting samples at five moisture contents to three levels of compaction: British Standard (Heavy) Compaction, 4.5 kg rammer method, British Standard (Light) Compaction, 2 5 kg rammer method, and an intermediate level 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 British Standard (Light) Compaction Test is specified and the subgrade moisture content was estimated to be 20 per cent.

If saturated subgrade conditions are anticipated, the compacted samples for the CBR test should be saturated by immersion in water for four days before being tested. In all other cases when CBR is determined by direct measurement, the CBR samples should not be immersed since this results in over design.

In areas where existing roads have been built on the same subgrade, direct measurements of the subgrade strengths can be made using a dynamic cone penetrometer (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.

Whichever method is used to obtain the subgrade strength, each sample or each test will usually give different results and these can sometimes cover a considerable range. For design purposes it is important that the strength of the subgrade is not seriously underestimated for large areas of pavement or overestimated to such an extent that there is a risk of local failures. The best compromise for design purposes is to use the lower ten percentile value i.e. that value which is exceeded by 90 per cent of the readings. The simplest way to obtain this is to draw a cumulative frequency distribution of strength as shown in Figure 4. If the characteristics of the subgrade change significantly over sections of the route, different subgrade strength values for design should be calculated for each nominally uniform section.

The structural catalogue requires that the subgrade strength for design is 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 CBR's less than 2, special treatment is required which is not covered in this Road Note.

TABLE 3.1

Subgrade strength classes

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

If equipment for carrying out laboratory compaction and CBR tests is not available, a less precise estimate of the minimum subgrade strength class can be obtained from Table 3 2. 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 British Standard (Light) Compaction Test, 2.5 kg rammer method. The Table is appropriate for subgrade moisture Categories (1) and (2) but can be used for Category (3) if conservative strength estimates are acceptable.

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

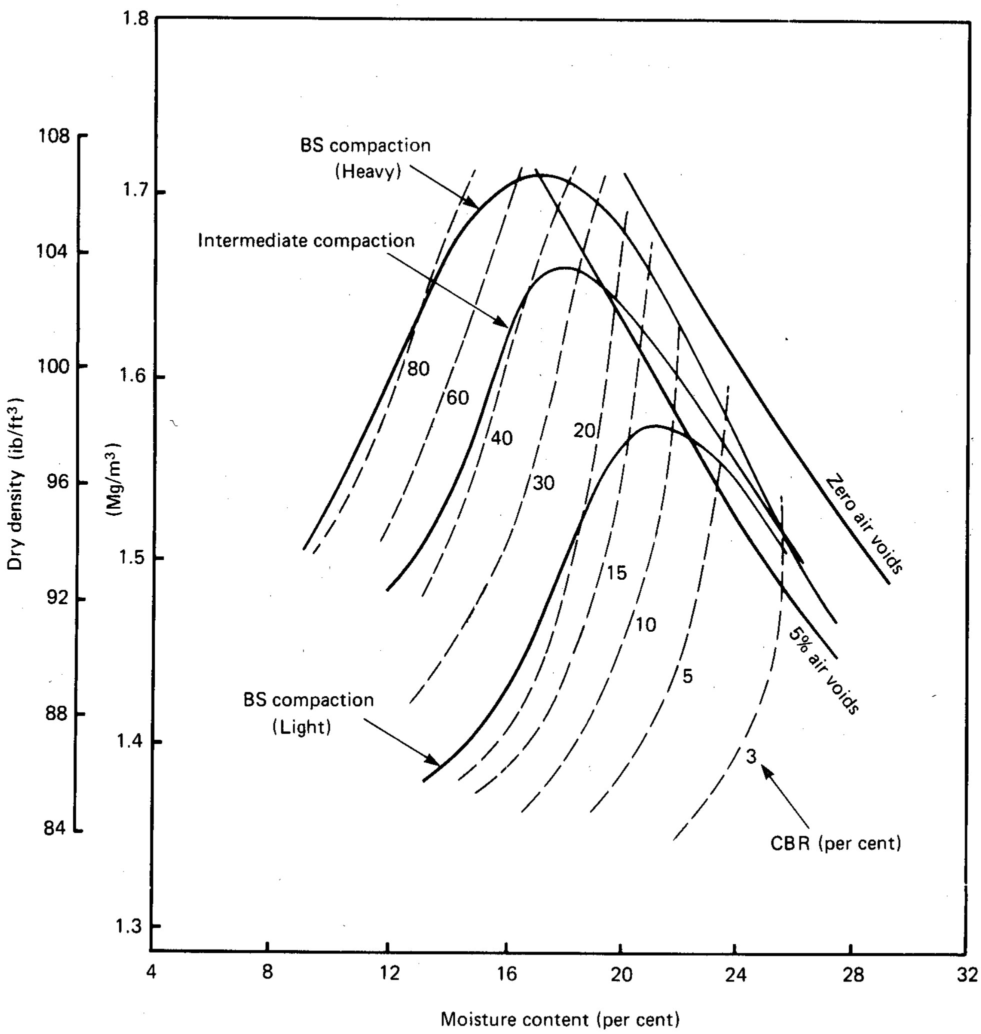


Fig.3 Dry density-moisture content-CBR relationships for sandy-clay soil

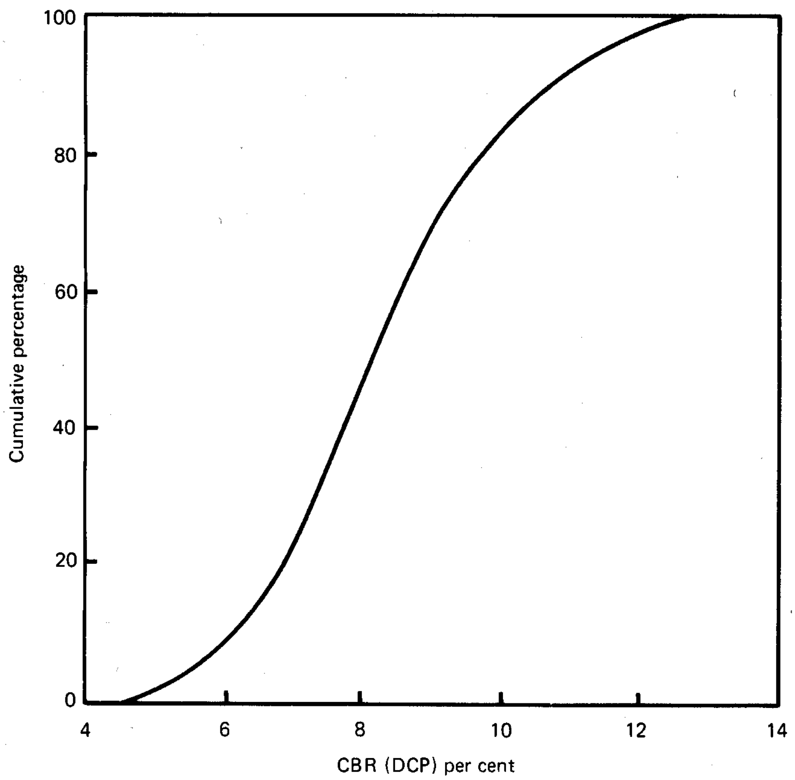


Fig.4 Distribution of subgrade strength

TABLE 3.2

Estimated design subgrade strength class under sealed roads in the presence of a water table

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

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

Notes. 1. 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.

2. Table 3.2 is not applicable for silt, micaceous, organic or tropically weathered clays. Laboratory CBR tests should be undertaken for these soils.

4. EMBANKMENTS AND CUTTINGS

4.1 EMBANKMENTS

4.1.1 Introduction and survey

Embankments and cuttings will be required to obtain a satisfactory alignment on all but the lowest standard of road Embankments will be needed (i) to raise the road above flood water levels, (ii) in sidelong ground, (iii) across gullies and (iv) at the approaches to water crossings. High embankments impose a considerable load on the underlying soil and settlement should always be expected. Some transported soils are particularly susceptible, wind-blown sands and unconsolidated estuarine soils being particular examples. Organic material decays quickly above the water table and such soils are formed only in marshy areas where decay below the water level proceeds slowly. The residual soils that are widespread in the tropics are not usually compressible and any settlement which does occur is likely to be substantially complete by the time the embankment is constructed. An exception to this is the halloysitic clays developed from volcanic ash whose fragile structure can be broken down causing collapse under embankment loads. Potentially compressible soils should be identified at the survey stage which precedes new construction.

During the survey it is also essential to look for evidence of water flow across the line of the road, either on the surface or at shallow depth. Temporary, perched water tables are common within residual soils and may not be readily apparent in the dry season. Drains must be installed to intercept ground water, and culverts of suitable size must be provided to allow water to cross the road alignment where necessary.

It is also important to identify any areas of potential ground instability which might affect embankments. Particular care is required in gullies, which themselves may be indicative of weakness in the geological structure, but steep side-sloping ground may also be suspect and evidence of past soil movement should be sought.

Evidence of past (dormant) instability is revealed by a range of slope features. On the surface, springs or patches of reeds or sedges are a sign that the slope may become saturated during the rainy season. Trees leaning at different angles (especially upslope) are a sign of disturbance by ground movement. However, it should be noted that trees leaning outwards (downslope), all at a similar angle, are usually not so much a sign of instability as a sign that the trees have grown at an angle to seek light. The age of trees can indicate former movement if they are all of a similar youthful age and there are no old trees present, this suggests that regeneration has taken place following a recent slide.

The shape of the ground itself is a good indicator of past movement. The classic features of hummocky ground (irregular, pocketed surface), cracks and small ponds are signs of a deep-seated landslide mass. Another sign is the presence on the slope of hollow bowl-shaped depressions with a steep head, curved in plan, which may represent the head area of old slips. They can be of any size, from a few tens of metres across to several hundred metres.

Gullies that are active can put embankments at risk by bringing down debris, blocking the culvert and damming up against the embankment. An active gully carries a heavy load of material, typically of mixed sizes including sand and fines. Stable gullies generally contain only boulder and cobble sized material (the smaller sizes wash away), and may bear vegetation more than one year old in the gully floor. If the gully sides are being undercut by the stream and loose sediment moving in the gully floor, fresh debris will be brought into the gully, making the situation downstream worse.

The slope below the road should be examined to ensure that it is not being undercut by the stream at the base. If this is happening, the whole embankment and road are at risk from slope failure expanding upwards from below.

Evidence of slope instability is not easy to detect in trial pits because soils on steep slopes are often disturbed by slow creep under gravity, resulting in a jumbled soil profile. However, former slope movement is sometimes indicated by ancient organic horizons (buried soils) lying parallel to the present surface, or by clayey horizons lying parallel to the surface, that represent old sliding surfaces. Water often travels along these. The bedrock, too, can indicate a danger of movement. Rocks whose bedding lies parallel to the hillside, or dips out of the hillside, are prone to failure along the bedding plane, as are rocks containing joint surfaces (parallel planes of weakness) oriented this way. Weak, weathered and highly fractured rocks all represent a hazard, especially if the fissures are open, showing that the rock mass is dilating under tension.

In steep side-sloping ground where the slope exceeds 1 in 6, it is normal practice to cut horizontal benches into the slope to simplify construction and to help key the embankment to the slope. At the same time, internal drainage is usually installed to remove sub-surface water from within the structure.

Problems with embankments are fortunately rare but when they occur the consequences can be serious. It is therefore important that all potential problems are identified during the survey and recommendations made for more detailed investigations where necessary. Such investigations are expensive and need to be planned systematically, with additional testing and expert advice being commissioned only as required. An example of such an approach has been described by NITRR (1987a).

4.1.2 Materials

Almost all types of soil, ranging from sandy clays through to broken rock, can be used for embankment construction, the main limitation being the ease with which the material can be handled and compacted. The embankment material will usually be obtained from borrow areas adjacent to the road or hauled from nearby cuttings. Material of low plasticity is preferred because such material will create fewer problems in wet weather. With more plastic soils, greater care is necessary to keep the surfaces shaped and compacted so that rain water is shed quickly. If the embankment is higher than about 6 metres, it is desirable to reserve material of low plasticity for the lower layers.

4.1.3 Design

Side slopes for high embankments should normally be between 1 in 1.5 and 1 in 2 (vertical: horizontal) Variations from this slope for local soils and climates are more reliably derived from local experience than from theoretical calculations. Slacker slopes are sometimes desirable for silty and clayey soils, especially in wet climates. In all cases it is important to protect the side slopes from the erosive action of rain and wind. Usually this should be done by establishing a suitable cover of vegetation (Howell et al (1991)) but granular materials will be needed in arid areas.

Particular care is needed with expansive soils, especially those containing montmorillonite. If construction in such soils cannot be avoided, earthworks must be designed to minimise subsequent changes in moisture content and consequent volume changes. For example, the soil should be placed and compacted at a moisture content close to the estimated equilibrium value and it may also be advantageous to seal the road shoulders with a surface dressing. On low embankments in expansive soils, relatively shallow side slopes should be used i e 1 in 3, and these should be covered with well graded granular material. Nevertheless some volume changes must be expected with expansive soils and any cracks which develop, either in the side slopes or shoulders, should be sealed before water enters the structure.

When the subgrade is a particularly expansive soil, it may be necessary to replace the expansive material with non-expansive impermeable soil to the depth affected by seasonal moisture changes.

4.1.4 Construction over compressible soils

Transported soils. In the design of embankments over compressible soils, it is necessary to determine the amount of settlement which will occur and ensure that the rate of loading is sufficiently slow to prevent pore water pressures from exceeding values at which slip failures are likely to occur. A reasonably accurate estimate of total settlement can be obtained from consolidation tests as outlined by Lewis et al (1975) and TRRL (1976) but the theory usually overestimates the time required for settlement to occur. This is because most deposits of unconsolidated silt or clay soils contain horizontal lenses of permeable sandy sod which allow water to escape.

High pore water pressures can be detected using piezometers set at different depths This often provides a reliable method of estimating the time required for consolidation and also provides a means of checking that pore water pressures do not reach unacceptably high levels during construction. Further precautions can be taken by installing inclinometers to detect any movement of soil which might indicate that unstable conditions exist.

If necessary, consolidation can be accelerated by installing some form of vertical drainage. Sand drains consisting of columns of sand of about 500 mm diameter set at regular intervals over the area below the embankment have been used successfully but nowadays wick drains are more common. If the embankment is sufficiently stable immediately after construction, the rate of consolidation can be increased by the addition of a surcharge of additional material which is subsequently removed before the pavement is constructed.

Organic soils. Organic soils are difficult to consolidate to a level where further settlement will not occur, and they provide a weak foundation even when consolidated. It is therefore best to avoid such materials altogether. If this is not possible, they should be removed and replaced If neither of these options is feasible, and provided soil suitable for embankments is available, methods of construction similar to those adopted for unconsolidated silt-clays should be used.

4.1.5 Compaction of embankments

Uniformity of compaction is of prime importance in preventing uneven settlement. Although some settlement can be tolerated it is important that it is minimised, especially on the approaches to bridges and culverts where adequate compaction is essential.

In the United Kingdom, compaction requirements are usually specified by means of a method specification which eliminates the need for in situ density tests (Department of Transport (1986)). In tropical countries it is more usual to use an end-product specification. It is therefore essential that laboratory tests are carried out to determine the dry density/moisture content relationships for the soils to be used and to define the achievable densities. In the tropics the prevailing high temperatures 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 plant in use.

The upper 500 mm of soil immediately beneath the subbase or capping layer i e the top of the embankment fill or the natural subgrade, should be well compacted In practice this means that a minimum level of 93-95 per cent of the maximum dry density obtained in the British Standard (Heavy) Compaction Test, 4 5 kg rammer should be specified (a level of 98 per cent is usually specified for roadbases and sub-bases). 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 in the British Standard (Heavy) Compaction Test, 4.5 kg rammer, is suitable, or a level of 95-100 per cent of the maximum density obtained in the lighter test using the 2.5

kg yammer. The British Standard Vibrating Hammer Test (BS 1377, Part 4 (1990)) should be used for non-cohesive soils and a level of 90-93 per cent of maximum density should be specified for the lower layers and 95 per cent for the upper layers. Compaction trials should always be carried out to determine the best way to achieve the specified density with the plant available (Parsons (1993)).

In and areas where water is either unavailable or expensive to haul, the dry compaction techniques developed by O'Connell et al (1987) and Ellis (1980) should be considered. Figure 5 illustrates that high densities can be achieved at low moisture contents using conventional compaction plant, and field trials have shown that embankments can be successfully constructed using these methods.

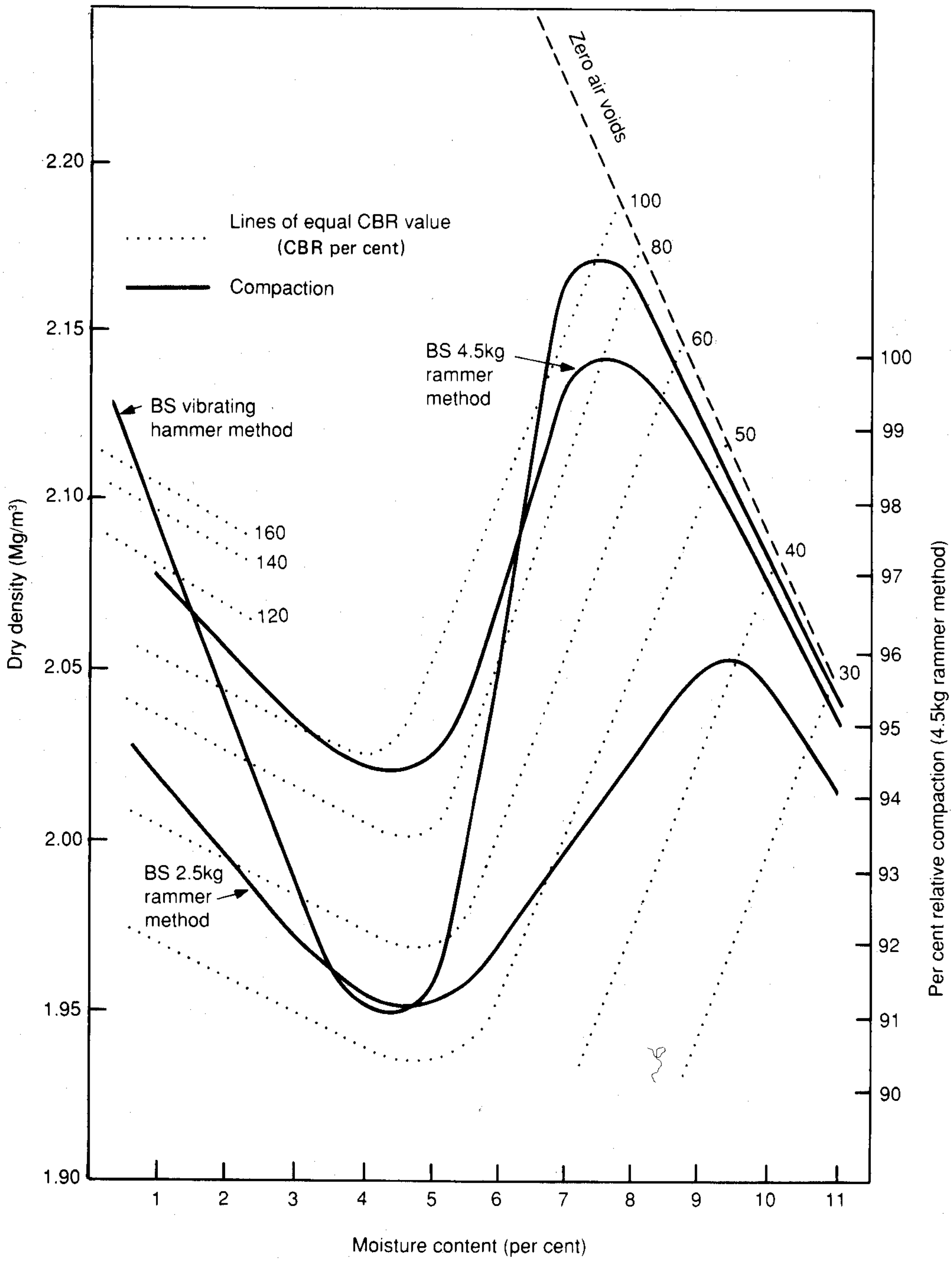


Fig.5 Dry density-moisture content relationships for a gravel-sand-clay

4.1.6 Site control

It is not easy to obtain an accurate measure of field density on site. The standard methods of measurement are tedious and not particularly reproducible Furthermore, most soils are intrinsically variable in their properties and it is difficult to carry out sufficient tests to define the density distribution. An acceptable approach to this problem is to make use of nuclear density and moisture gauges. Such devices are quicker and the results are more reproducible than traditional methods, but 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 but improvements in nuclear techniques are always being made and trials should be carried out for each situation.

Additional site control can be exercised by making use of the Moisture Condition Test developed by Parsons (Parsons(1976), Parsons and Toombs (1987)). This test provides a rapid method of determining the moisture condition of soils and its use is described in BS 1377, Part 4 (1990).

4.2 CUTTINGS

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

4.2.1 Slope stability

Methods of analysing 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 analysis (Bishop (1955)). 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 soil cuttings.

4.2.2 Surveys

The construction of cuttings invariably disturbs the natural stability of the ground by the removal of lateral support and a change in the natural ground water conditions. The degree of instability will depend on the dip and stratification of the soils relative to the road alignment, the angle of the slopes, the ground water regime, the type of material, the dimensions of the cut, and numerous other variables. A full investigation is therefore an expensive exercise but, fortunately, most cuttings are small and straightforward Investigations for the most difficult situations are best left to specialists and guidance on the need for this can be obtained in, for example, publications of the NITRR (1987b). Such guidance is defined by local experience and every opportunity should be taken to maintain a local data base.

An integral component of a survey is to catalogue the performance of both natural and man-made slopes in the soils encountered along the length of the road and to identify the forms of failures to inform the design process and to make best use of the empirical evidence available in the area. These procedures have been described in detail by Anderson and Lloyd (1991) and considerable future advantages can be obtained if the field experience is encapsulated in the land classification procedures described by Lawrance et al (1993).

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 water courses crossing the road line must be identified and the need for culverts and erosion control established.

4.2.3 Design and construction

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

A similar problem applies to the use of cut-off drains at the top of the cutting which are designed to prevent run-off 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 cutting 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 and specialist advice is recommended.

As with embankments, it is essential that provision is made to disperse surface water from the formation at all stages of construction. Temporary formation levels should always be maintained at a slope to achieve this. Drainage is critically important because pore water pressures created by the available head of ground water in the side slopes can cause rapid distress in the pavement layers. Subsoil drains at the toe of the side slopes may be necessary to alleviate this problem.

The subsequent performance, stability and maintenance of cuttings will depend on the measures introduced to alleviate the problems created by rainfall and ground water Invariably it is much more cost effective to install all the necessary elements at construction rather than to rely on remedial treatment later. Further guidance can be obtained from standard textbooks and reference books (Paige-Green (1981), Bell (1987)).

5. DRAINAGE AND ROAD SHOULDERS

5.1 THE DRAINAGE SYSTEM

One of the most important aspects of the design of a road is the provision 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 drainage system, which is properly maintained, is vital to the successful operation of a road and the road designs described in this Note are based on the assumption that the side drains and culverts associated with the road are properly designed and function correctly.

Drainage within the pavement layers themselves is an essential element of structural design because the strength of the subgrade used for design purposes depends on the moisture content during the most likely adverse conditions. 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 dram away quickly from within the pavement layers.

5.2 PAVEMENT CROSS-SECTION

The width of the carriageway and the overall geometric design of the road are dealt with in Overseas Road Note No 5 (Transport and Road Research Laboratory (1988)). For design traffic volumes in excess of about 1000 vehicles per day, carriageway widths of at least 7 metres should be used throughout and additional lanes will be needed when the capacity of a two-lane road is exceeded.

Shoulders are an essential element of the structural design of a road, providing lateral support for the pavement layers. They are especially important when unbound materials are used in the pavement and for this type of construction it is recommended that shoulders should be at least 2 metres wide. For bound roadbases, shoulder width can be reduced if required If there is a large volume of non-motorised traffic, then shoulder width should be increased to a minimum of 3 metres. In order to exclude water from the road, the top of the shoulders should be impermeable and a surface dressing or other seal may need to be applied (see Chapter 9). Unsurfaced shoulders are not generally recommended because they often require considerable maintenance if satisfactory performance is to be guaranteed. Sealed shoulders also prevent the ingress of water at the edge of the pavement, which is an area particularly vulnerable to structural Damage. Shoulders should be differentiated from the carriageway e.g by the use of edge markings, different sized aggregate or different coloured aggregate.

Crossfall is needed on all roads in order to assist the shedding of water into the side drains. A suitable value for paved roads is about 3 per cent for the carriageway, with a slope of about 4-6 per cent for the shoulders. An increased crossfall for the carriageway e g 4 per cent, is desirable if the quality of the final shaping of the road surface is likely to be low for any reason.

There is evidence that there are also benefits to be obtained by applying steeper crossfalls to layers at successive depths in the pavement. The top of the sub-base should have a crossfall of 3-4 per cent and the top of the subgrade should be 4-5 per cent. These crossfalls not only improve the drainage performance of the various layers, but also provide a slightly greater thickness of material at the edge of the pavement where the structure is more vulnerable to damage. The design thickness should be that at the centre line of the pavement.

5.3 DRAINAGE OF LAYERS

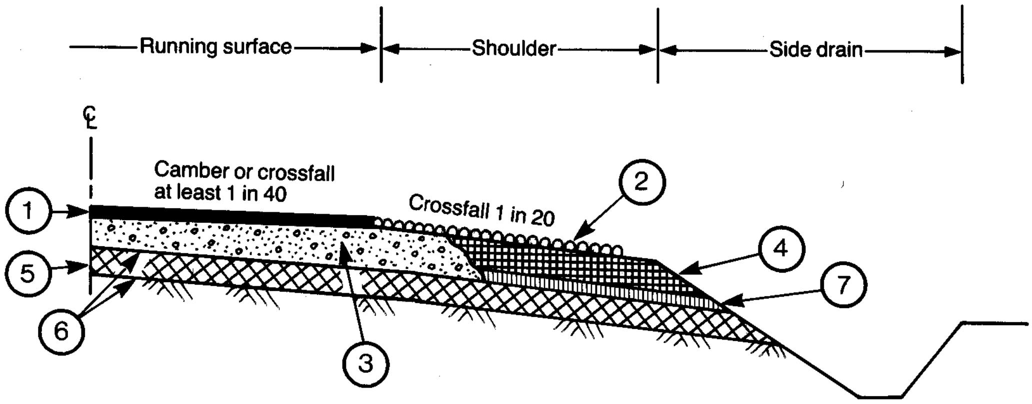
Provided the crossfalls indicated above are adhered to and the bituminous surfacing and the shoulders are properly maintained, rainwater falling on the road will be shed harmlessly over the shoulders. When permeable roadbase materials are used (see Section 3 1), particular attention must be given to the drainage of this layer. Ideally, the roadbase and sub-base should extend right across the shoulders to the drainage ditches as shown in Fig 6. Under no circumstances should the `trench' type of cross-section be used in which the pavement layers are confined between continuous impervious shoulders.

If it is too costly to extend the roadbase and sub-base material across the shoulder, drainage channels at 3m to 5m intervals should be cut through the shoulder to a depth of 50 mm below sub-base level. These channels should be back-filled with material of roadbase quality but which is more permeable than the roadbase itself, and should be given a fall of 1 in 10 to the side ditch. Alternatively a continuous drainage layer of pervious material of 75 mm to 100 mm thickness can be laid under the shoulder such that the bottom of the drainage layer is at the level of the top of the sub-base. This latter is by far the better of the two alternatives.

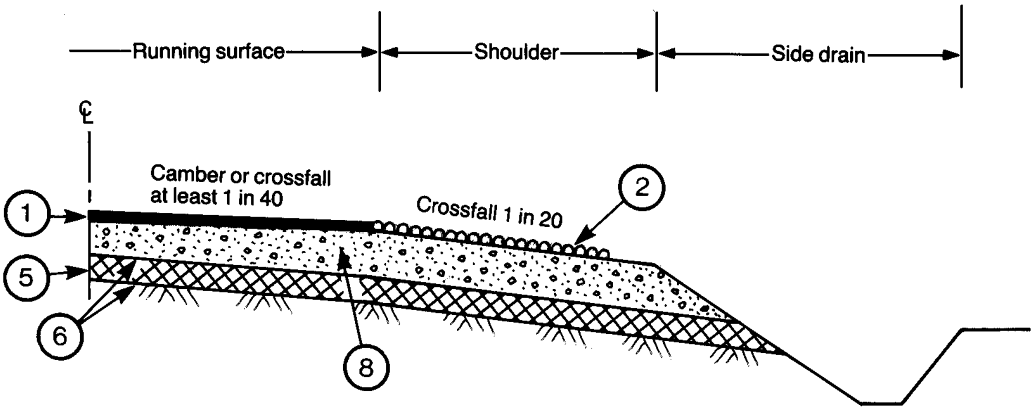
In circumstances where the subgrade itself is permeable and can dram freely, it is preferable that vertical drainage is not impeded. If this can be done by ensuring that each layer of the pavement is more permeable than the layer above, then the additional drainage layer through the shoulders (layer No 7 in Fig. 6) is not required.

5.4 SHOULDER MATERIALS

Although the ideal solution is to extend the roadbase and sub-base outwards to form the shoulders, when the roadbase material is non-plastic it may lack sufficient cohesion to withstand the abrasive action of traffic unless



(a) Separate shoulder material



(b) Extended roadbase and sub-base

1 Impervious surfacing
2 Shoulders surface dressed (giving contrasting texture to running surface)
3 Roadbase extending under shoulder for at least 500mm
4 Shoulder material capable of supporting occasional traffic
5 Impervious sub-base carried across full width of construction
6 Formation and sub-base constructed with crossfall of 1 in 30 (providing drainage path for any water that enters and also a thicker and stronger pavement on the outside wheel track)
7 Drainage layer of pervious material
8 Roadbase extending through shoulder

Fig.6 Cross section of road showing drainage arrangements

it is sealed with a surface dressing (see Section 9.2.1). In circumstances where extending the roadbase is not possible and the shoulder is not to be sealed, the shoulder material should be selected using the same principles as for a gravel-surfaced road or a sub-base to carry construction traffic. Thus the material should be strong enough to carry occasional vehicles and should be as cohesive as possible without being too weak when wet. The material will normally be of sub-base quality and the soaked CBR value at the specified density should exceed 30 per cent, except perhaps in and areas where the binding action of plastic fines may become a more important criterion (see Section 6 2.2, Tables 6.6 and 6 7 for general guidance).

It is also very desirable if at least the outer edge of the shoulder is able to support the growth of grasses which help to bind the surface and prevent erosion. On rural roads where the shoulders rarely need to carry traffic, excellent shoulder performance can be obtained if the whole of the shoulder is grassed In these circumstances it is necessary for this grass to be cut regularly to prevent the level of the shoulder building up above the level of the carriageway and thereby causing water to be retained at the carriageway-shoulder interface where it can penetrate the road structure and cause structural weakening.

6. UNBOUND PAVEMENT MATERIALS

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

6.1 ROADBASE MATERIALS

A wide range of materials can be used as unbound roadbases including crushed quarried rock, crushed and screened, mechanically stabilised, modified or naturally occurring `as dug' gravels. Their suitability for use depends primarily on the design traffic level of the pavement and climate but all roadbase 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 types of roadbase are suitable, 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 Ti and T2). Their use should be based on the results of performance studies and should incorporate any special design features which ensure their satisfactory performance. As a cautionary note, when considering the use of natural gravels a statistical approach should be applied in interpreting test results to ensure that their inherent variability is taken into account in the selection process.

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

6.1.1 Crushed stone

Graded crushed stone (GB 1,A and GB1,B). *Two* types of material are defined in this category. One is produced by crushing fresh, quarried rock (GB1,A) 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. The other is derived from crushing and screening natural granular material, rocks or boulders (GB1,B) and may contain a proportion of natural, fine aggregate. Typical grading limits for these materials are shown in Table 6 2. After crushing, the material should be angular in shape with a Flakiness Index (British Standard 812, Part 105 (1990)) of less than 35 per cent. 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 roadbase, the aim should be to achieve maximum impermeability compatible with good compaction and high stability under traffic.

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 (1990)) 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 climate. Other simpler tests e.g. the Aggregate Impact

TABLE 6.1

Properties of unbound materials

Code	Description	Summary of specification
GB1,A	Fresh, crushed rock	Dense graded, unweathered crushed stone, non-plastic parent fines
GB1,B	Crushed rock, gravel or boulders	Dense grading, PI < 6, soil or parent fines
GB2,A	Dry-bound macadam	Aggregate properties as for GB1,B (see text), PI < 6
GB2,B	Water-bound macadam	Aggregate properties as for GB1,B (see text), PI < 6
GB3	Natural coarsely graded granular material including processed and modified gravels	Dense grading, PI < 6 CBR after soaking > 80
GS	Natural gravel	CBR after soaking > 30
GC	Gravel or gravel-soil	Dense graded, CBR after soaking > 15

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

TABLE 6.2

Grading limits for graded crushed stone roadbase materials (GB1,A; GB1,B)

BS test sieve (mm)	Percentage by mass of total aggregate passing test sieve		
	Nominal maximum particle size		
	37.5 mm[1]	28 mm	20 mm
50	100	-	-
37.5	95 - 100	100	-
28	-	-	100
20	60 - 80	70 - 85	90 - 100
10	40 - 60	50 - 65	60 - 75
5	25 - 40	35 - 55	40 - 60
2.36	15 - 30	25 - 40	30 - 45
0.425	7 - 19	12 - 24	13 - 27
0.075[2]	5 - 12	5 - 12	5 - 12

Notes 1. Corresponds approximately to the UK specification for wet-mix macadam (Department of Transport, 1986)
2. For paver-laid materials a lower fines content may be accepted.

TABLE 6.3

Mechanical strength requirements for the aggregate fraction of crushed stone roadbases (GB1,A; GB1,B) as defined by the Ten Per Cent Fines Test

Climates	Typical annual rainfall (mm)	Minimum 10% fines values (kN)	Minimum ratio wet/dry test (%)
Moist tropical, wet tropical and seasonally wet tropical	>500	110	75
Arid and semi-arid	<500	110	60

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.

When dealing with materials originating from the weathering of basic igneous rocks the recommendations in Section 6.1 2 should be used.

The fine fraction of a GB1,A material should be non-plastic. For GB1,B materials the maximum allowable PI is 6. When producing these materials, the percentage passing the 0.075 mm sieve should be chosen according to the grading and plasticity of the fines. For materials with non-plastic fines, the proportion passing the 0.075 mm sieve may approach 12 per cent 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 Plasticity Product (PP) value of 45 is recommended where

PP = PI x (percentage passing the 0.075 mm sieve)

In order to meet these requirements it may be necessary to add a low proportion of hydrated lime or cement to alter the properties of the fines. Such materials are commonly referred to as modified materials Further details are given in Chapter 7.

These materials may be dumped and spread by grader but it is preferable to use a payer 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 per cent of the maximum dry density obtained in the British Standard (Heavy) Compaction Test, 4 5 kg rammer, or the British Standard Vibrating

Hammer Test (British Standard 1377, Part 4 (1990)). The compacted thickness of each layer should not exceed 200 mm.

When properly constructed, crushed stone roadbases will have CBR values well in excess of 100 per cent. In these circumstances there is no need to carry out CBR tests.

Dry-bound macadam (GB2,A). Dry-bound macadam is a traditional form of construction, formerly used extensively in the United Kingdom, and is comparable in performance with a graded crushed stone It has been used successfully in the tropics and is particularly applicable in areas where water is scarce or expensive to obtain It is also suitable where labour-intensive construction is an economic option. The materials consist of nominal single-sized crushed stone and non-plastic fine aggregate (passing the 5.0 mm sieve). The fine material should preferably be well graded and consist of crushed rock fines or natural, angular pit sand.

The dry-bound macadam process involves laying single-sized crushed stone of either 37.5 mm or 50 mm nominal size in a series of layers to achieve the design thickness. The compacted thickness of each layer should not exceed twice the nominal stone size. Each layer of coarse aggregate should be shaped and compacted and then the fine aggregate spread onto the surface and vibrated into the interstices to produce a dense layer. Any loose material remaining is brushed off and final compaction carried out, usually with a heavy smooth-wheeled roller. This sequence is then repeated until the design thickness is achieved. To aid the entry of the fines, the grading of the 37 5 mm nominal size stone should be towards the coarse end of the recommended range. Economy in the production process can be obtained if layers consisting of 50 mm nominal size stone and layers of 37 5 mm nominal size stone are both used to allow the required total thickness to be obtained more precisely and to make better overall use of the output from the crushing plant.

Water-bound macadam (GB2,B). Water-bound macadam is similar to dry-bound macadam. It consists of two components namely a relatively single-sized stone with a nominal maximum particle size of 50 mm or 37 5 mm and well graded fine aggregate which passes the 5.0 mm sieve. The coarse material is usually produced from quarrying fresh rock. The crushed stone is laid, shaped and compacted and then fines are added, rolled and washed into the surface to produce a dense material. Care is necessary in this operation to ensure that water sensitive plastic materials in the sub-base or subgrade do not become saturated. The compacted thickness of each layer should not exceed twice the maximum size of the stone The fine material should preferably be non-plastic and consist of crushed rock fines or natural, angular pit sand.

Typical grading limits for the coarse fraction of GB2A or GB2B materials are given in Table 6 4. The grading of M2 and M4 correspond with nominal 50 mm and 37.5 mm single-sized roadstones (British Standard 63 (1987)) and are appropriate for use with mechanically crushed aggregate. M1 and M3 are broader specification M1 has been used for hand-broken stone but if suitable screens are available, M2, M3 and M4 are preferred.

Aggregate hardness, durability, particle shape and in situ density should each conform to those given above for graded crushed stone.

6.1.2 Naturally occurring granular materials

Normal requirements for natural gravels and weathered rocks (GB3). A wide range of materials including latentic, calcareous and quartzitic gravels, river gravels and other transported gravels, or granular materials resulting from the weathering of rocks can be used successfully as roadbases. Table 6.5 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

TABLE 6.4

Typical coarse aggregate gradings for dry-bound (GB2,A) and water-bound macadam (GB2,B)

BS test sieve (mm)	Percentage by mass of total aggregate passing test sieve			
	M1	M2(1)	M3	M4(2)
75	100	100	100	-
50	85 - 100	85 - 100	85 - 100	100
37.5	35 - 70	0 - 30	0 - 50	85 - 100
28	0 - 15	0 - 5	0 - 10	0 - 40
20	0 - 10	-	-	0 - 5

Notes. 1. Corresponds to nominal 50 mm single-sized roadstone.
2. Corresponds to nominal 37.5 mm single-sized roadstone. To aid the entry of fines, the coarser end of this grading is preferred.

TABLE 6.5

Recommended particle size distributions for mechanically stable natural gravels and weathered rocks for use as roadbases (GB3)

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

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.

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, aggregate hardness criteria based on a minimum soaked Ten Per Cent Fines Value of 50kN or a maximum soaked Modified Aggregate Impact Value of 40 may be specified (British Standard 812, Part 112 (1990)).

The fines of these materials should preferably be non-plastic but should normally never exceed a PI of 6 As an alternative to specifying PI, a Linear Shrinkage not exceeding 3 may be specified.

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

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

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

When used as a roadbase, the material should be compacted to a density equal to or greater than 98 per cent of the maximum dry density achieved in the British Standard (Heavy) Compaction Test, 4.5 kg rammer. When compacted to this density in the laboratory, the material should have a minimum CBR of 80 per cent after four days immersion in water (British Standard 1377, Part 4 (1990)).

Arid and semi-arid areas. In low rainfall areas in the tropics, typically with a mean annual rainfall of less than 500 mm, and where evaporation is high, moisture conditions beneath a well sealed surface are unlikely to rise above the optimum moisture content determined in the British Standard (Heavy) Compaction Test. In such conditions, high strengths (CBR>80 per cent) 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 (TI, T2) the maximum allowable PI can be increased to 12 and the minimum soaked CBR criterion reduced to 60 per cent 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 gains entry into the pavement and this can lead to rapid and premature failure. The state of decomposition also affects their long term durability when stabilised 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 (British Standards Institution (1975) and Weinert (1980)). 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, the use of various chemical soundness tests e.g. sodium or magnesium sulphate (British Standard 812 Part 121 (1990)), the use of dye adsorption tests (Sameshima and Black (1979)) or the use of a modified Texas Ball Mill Test (Sampson and Netterberg (1989)). Indicative limits based on these tests include (i) a maximum secondary mineral content of 20 per cent, (ii) a maximum loss of 12 or 20 per cent after 5 cycles in the sodium or magnesium sulphate tests respectively (iii) a Clay Index of less than 3 and (iv) a Durability Mill Index of less than 90. 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.

Materials of marginal quality. In many parts of the world, as-dug gravels which do not normally meet the normal specifications for roadbases have been used successfully. They include latentic, 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. Successful use often depends on specific design and construction features. It is not possible to give general guidance on the use of all such materials and the reader is advised to consult the appropriate source references (e g. CIRIA (1988), Lionfanga et al (1987), Netterberg and Pinard (1991), Newill et al (1987) and Rolt et al (1987)).

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 per cent 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.

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 roadbase. Under special circumstances it may also act as a filter or as a drainage layer. In wet climatic conditions, the most stringent requirements are dictated by the need to support construction traffic and paving equipment. In these circumstances the sub-base material needs to be more tightly specified In dry climatic conditions, in areas of good drainage, and where the road surface remains well sealed, unsaturated moisture conditions prevail and sub-base specifications may be relaxed. The selection of sub-base materials will therefore depend on the design function of the layer and the anticipated moisture regime, both in service and at construction.

6.2.1 Bearing Capacity

A minimum CBR of 30 per cent is required at the highest anticipated moisture content when compacted to the specified field density, usually a minimum of 95 per cent of the maximum dry density achieved in the British Standard (Heavy) Compaction Test, 4.5 kg rammer. 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 British Standard (Light) Compaction Test, 2.5 kg rammer. In such conditions, the sub-base material should be tested in the laboratory in an unsaturated state. Except in and areas (Category (3) in Chapter 3), if the roadbase allows water to drain into the lower layers, as may occur with unsealed shoulders and under conditions of poor surface maintenance where the roadbase 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 6.6 and 6.7 will usually be found to have adequate bearing capacity.

6.2.2 Use as a construction platform

In many circumstances the requirements of a sub-base are governed by its ability to support construction traffic without excessive deformation or ravelling. A high quality sub-base is therefore required where loading or climatic conditions during construction are severe. Suitable material should possess properties similar to those of a good surfacing material for unpaved roads. The material should be well graded and have a plasticity index at the lower end of the appropriate range for an ideal unpaved road wearing course under the prevailing climatic conditions. These considerations form the basis of the criteria given in Tables 6.6 and 6.7. 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.

6.2.3 Sub-base as a filter or separating layer

This may be required to protect a drainage layer from blockage by a finer material or to prevent migration of fines and the mixing of two layers. The two functions are similar except that for use as a filter the material needs to be capable of allowing drainage to take place and therefore the amount of material passing the 0 075 mm sieve must be restricted.

TABLE 6.6

Recommended plasticity characteristics for granular sub-bases (GS)

Climate	Liquid Limit	Plasticity Index	Linear Shrinkage
Moist tropical and wet tropical	<35	<6	<3
Seasonally wet tropical	<45	<12	<6
Arid and semi-arid	<55	<20	<10

TABLE 6.7

Typical particle size distribution for sub-bases (GS) which will meet strength requirements

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

The following criteria should be used to evaluate a sub-base as a separating or filter layer.

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

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

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

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

c) The ratio $\frac{Q15(\text{coarse layer})}{D15(\text{fine layer})}$ should lie between 5 and 40

These criteria may be applied to the materials at both the road base/sub-base and the sub-base/subgrade interfaces. Further details can be obtained in the appropriate references e.g. (NAASRA (1983)).

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. 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 British Standard (Heavy) Compaction Test, 4 5 kg yammer 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 behaviour 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-STABILISED MATERIALS

7.1 INTRODUCTION

This chapter gives guidance on the manufacture and use of cement and lime-stabilised materials in roadbase, sub-base, capping and selected fill layers of pavements. The stabilising process involves the addition of a stabilising agent to the soil, intimate mixing with sufficient water to achieve the optimum moisture content, compaction of the mixture, and final curing to ensure that the strength potential is realised. The subject has been reviewed by Sherwood (1993).

Many natural materials can be stabilised 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 stabilisation.

Stabilisation 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.
- Resistance to erosion is increased.
- Materials in the supporting layer cannot contaminate the stabilised layer.
- The effective elastic moduli of granular layers constructed above stabilised layers are increased.
- Lime-stabilised 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 stabilised 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 stabilisation reactions are reversible and the strength of the layers can decrease.
- The construction operations require more skill and control than for the equivalent unstabilised material.

Methods of dealing with these problems are outlined in Section 7.7.

The minimum acceptable strength of a stabilised 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 minimise the risk of reflection cracking. Three types of stabilised layer have been used in the structural design catalogue and the strengths required for each are defined in Table 7.1.

7.2 THE STABILISATION 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 a highly alkaline). The quantity of lime required to satisfy these reactions is determined by the initial consumption of lime test (ICL), (British Standard 1924 (1990)).

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 which holds the soil particles and aggregates together (NITRR (1986)).

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

TABLE 7.1

Properties of cement and lime-stabilised materials

Code	Description	Unconfined compressive strength* (MPa)
CB1	Stabilised roadbase	3.0 - 6.0
CB2	Stabilised roadbase	1.5 - 3.0
CS	Stabilised sub-base	0.75 - 1.5

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

7.3 SELECTION OF TYPE OF TREATMENT

The selection of the stabiliser is based on the plasticity and particle size distribution of the material to be treated. The appropriate stabiliser can be selected according to the criteria shown in Table 7 2 adapted from NAASRA (1986).

Some control over the grading can be achieved by limiting the coefficient of uniformity to a minimum value of 5. The coefficient of uniformity is defined as the ratio of the sieve size through which 60 per cent of the material passes to the sieve size through which 10 per cent passes (D60/D10 in the nomenclature of Section 6 2 3). If the coefficient of uniformity lies below this value the cost of stabilisation will be high and the maintenance of cracks in the finished road could be expensive. 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 sods with low plasticity to make them suitable for stabilisation with lime. If the plasticity of the soil is high there are usually sufficient reactive clay minerals which can be readily stabilised 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.

If possible, the quality of the material to be stabilised should meet the minimum standards set out in Table 7.3. Stabilised layers constructed from these materials are more likely to perform satisfactorily even if they are affected by carbonation during their lifetime (Section 7.7 3). Materials which do not comply with Table 7 3 can

TABLE 7.2

Guide to the type of stabilisation likely to be effective

Type of stabilisation	Soil properties: *More than 25% passing the 0.075 mm sieve*			*Less than 25% passing the 0.075 mm sieve*		
	PI≤10	10<PI≤20	PI>20	PI≤6 PP≤60	PI≤10	PI > 10
Cement	Yes	Yes	*	Yes	Yes	Yes
Lime	*	Yes	Yes	No	*	Yes
Lime-Pozzolan	Yes	*	No	Yes	Yes	*

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

TABLE 7.3

Desirable properties of material before stabilisation

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

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

sometimes be stabilised but more additive will be required and the cost and the risk from cracking and carbonation will increase.

Some aspects of construction must also be considered in selecting the stabiliser. 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 stabilisation, with a faster curing period, is likely to be more suitable than lime stabilisation.

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

7.4 CEMENT STABILISATION

7.4.1 Selection of cement content

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

The minimum cement content, expressed as a percentage of the dry weight of soil, should exceed the quantity consumed in the initial ion exchange reactions. Until research into the initial consumption of cement (ICC) is completed it is recommended that the percentage of cement added should be equal to or greater than the ICL. If there is any possibility that the material to be stabilised is unsound e.g. weathered basic igneous materials, then the Gravel ICL Test (NITRR (1984)) is preferred. In this test the aggregate is ground up to release any active clay minerals and the total sample tested

The durability of the stabilised mixture which satisfies the strength requirements for the particular layer should also be assessed. Mixtures produced from sound materials complying with the minimum requirements of Table 7.3 can be assumed to be durable if they achieve the design strength. Mixtures produced from other materials should be checked using the wet-dry brushing test (ASTM (1987)) which gives a good indication of the likelihood that a stabilised material will retain adequate strength during its service life in a pavement (Paige-Green et al (1990)).

Additional stabiliser is normally incorporated to take account of the variability in mixing which occurs on site. If good control is exercised over the construction operations, an extra one per cent of stabiliser is satisfactory for this purpose.

7.4.2 Preparation of specimens

The optimum moisture content and the maximum dry density for mixtures of soil plus stabiliser are determined according to British Standard 1924 (1990) for additions of 2, 4, 6 and 8 per cent of cement. These specimens should be compacted as soon as the mixing is completed. Delays of the order of two hours occur in practice and changes taking place within the mixed material result in changes in their compaction characteristics. To determine the sensitivity of the stabilised materials to delays in compaction, another set of tests must be conducted after two hours have elapsed since the completion of mixing.

Samples for the strength tests should also be mixed and left for two hours 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 British Standard (Heavy) Compaction Test, 4.5 kg rammer. These samples are then moist cured for 7 days and soaked for 7 days in accordance with BS 1924.

Two methods of moist curing are described in the Standard. The preferred method is to seal the specimens in wax but if this is not possible they must be wrapped in cling film and sealed in plastic bags. The specimens should be maintained at 25°C during the whole curing and soaking period.

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.

If suitable moulds are not available to produce cube specimens then 200 mm x 100 mm cylinders, 115.5 mm x 105 mm cylinders or 127 mm x 152 mm cylinders may be used and the results multiplied by the following correction factors to calculate equivalent cube strengths.

Sample Type	Correction Factor
200 mm x 100 mm diameter	1.25
115.5 mm x 105 mm diameter	1 04
127 mm x 152 mm diameter	0 96

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

When the plasticity of the soil makes it difficult to pulverise and mix intimately with the cement, the workability can be improved by first pre-treating the sod with 2 to 3 per cent of lime, lightly compacting the mixture, and leaving it to stand for 24 hours. The material is then repulvensed and stabilised with cement. If this method is used, the laboratory design procedure is modified to include the pre-treatment phase before testing as described above.

7.5 LIME STABILISATION

7.5.1 Properties of lime-stabilised materials

When lime is added to a plastic material, it first flocculates the clay and substantially reduces the plasticity index. This reduction of plasticity is time dependent during the initial weeks, and has the effect of increasing the optimum moisture content and decreasing the maximum dry density in compaction. The compaction characteristics are therefore constantly changing with time and delays in compaction cause reductions in density and consequential reductions in strength and durability. The workability of the soil also improves as the soil becomes more friable. If the amount of lime added exceeds the ICL, the stabilised material will generally be non-plastic or only slightly plastic.

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

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

7.5.2 Types of lime

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

Quicklime is an excellent stabiliser 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 trafficabdity 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 (Elks (1974)) to determine the output possible from small kilns and to assess the suitability of lime produced without commercial process control for soil stabilisation. Small batch kilns have subsequently been used to produce lime for stabilised layers on major road projects.

7.5.3 Selection of lime content

The procedure for selecting the lime content follows the steps used for selecting cement content and should, therefore, be carried out in accordance with British Standard 1924 (1990). The curing period for lime-stabilised materials is 21 days of moist cure followed by 7 days of soaking.

In tropical and sub-tropical countries the temperature of the samples should be maintained at 25°C which is near to 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.

7.6 POZZOLANS

One of the primary sources of pozzolan is the pulvensed fuel ash (PFA) collected from the boilers of coal-fired electricity generating stations.

PFA is usually mixed with lime in the proportions of 1 of lime to 3 or 4 of PFA but ratios of 1 to 2 up to 1 to 10 are used. The proportion depends on the reactivity of the particular fly ash which varies substantially from source to source. Lime and fly ash treated layers have a similar performance to cement treated layers constructed from the same aggregate material. The final mixtures should be chosen after a series of laboratory tests carried out after 21 days of moist cure and 7 days of soaking to determine the optimum ratio of lime to fly ash and the optimum lime content (expressed as a percentage of dry soil).

In many tropical countries there are substantial quantities of bagasse (the fibrous residue from the crushing of sugar cane) and husks from rice. Both are rich in silica. When burnt, their ash contains a substantial amount of amorphous silica which reacts with lime (Cook and Suwanvitaya (1982), Mehta (1979)).

Lime and rice-husk ash mixtures gain strength quickly during the early period of curing but little additional strength is obtained after 28 days of moist curing. The long-term strength depends on the stability of the calcium silicate hydrates. Under certain conditions lime leaching can occur and eventually the strength will be reduced, but the presence of excess lime (free lime) can stabilise the calcium silicate hydrate. Mixtures of lime and rice-husk ash in the proportions 2:3 are the most stable and have the highest strength but the durability may be improved by increasing the lime content to give a 1:1 mixture.

7.7 CONSTRUCTION

7.7.1 General methodology

The construction of stabilised layers follows the same procedure whether the stabilising agent is cement, lime or mixtures of lime-pozzolan. After the surface of the layer has been shaped, the stabiliser is spread and then mixed through the layer. Sufficient water is added to meet the compaction requirements and the material mixed again. The layer must be compacted as soon as possible, trimmed, re-rolled and then cured. The effect of each operation on the design and performance of the pavement is discussed below.

Spreading the stabiliser. The stabiliser can be spread manually by `spotting' the bags at predetermined intervals, breaking the bags and then raking the stabiliser across the surface as uniformly as possible. Lime has a much lower bulk density than cement and it is possible to achieve a more uniform distribution with lime when stabilisers are spread manually. Alternatively, mechanical spreaders can be used to meter the required amount of stabiliser onto the surface.

Mixing. Robust mixing equipment of suitable power for the pavement layer being processed is capable of pulverising the soil and blending it with the stabiliser and water. The most efficient of these machines carry out the operation in one pass, enabling the layer to be compacted quickly and minimising the loss of density and strength caused by any delay in compaction. Multi-pass machines are satisfactory provided the length of pavement being processed is not excessive and each section of pavement can be processed within an acceptable time. Graders have been used to mix stabilised materials but they are inefficient for pulverising cohesive materials and a considerable number of passes are needed before the quality of mixing is acceptable. They are therefore very slow and should only be considered for processing lime-stabilised layers because of the greater workability of lime-stabilised materials and the subsequent diffusion of lime through the soil aggregations (Stocker (1972)).

Plant pre-mixing gives the possibility of better control than in-place spreading and mixing provided that the plant is close enough to the site to overcome possible problems caused by delays in delivery. This can often be justified by the lower safety margins on stabiliser content and target layer thicknesses that are possible.

Compaction. A stabilised layer must be compacted as soon as possible after mixing has been completed in order that the full strength potential can be realised and the density can be achieved without over stressing the material. If the layer is over stressed, shear planes will be formed near the top of the layer and premature failure along this plane is likely, particularly when the layer is only covered by a surface dressing.

Multi-layer construction. When two or more lifts are required to construct a thick layer of stabilised material, care must be taken to prevent carbonation at the surface of the bottom lift. It is also important that the stabiliser is mixed to the full depth of each layer. A weak band of any type can cause over stressing and premature failure of the top lift followed by deterioration of the lower section.

In general, the thickness of a lift should not be greater than 200 mm or less than 100 mm.

Care should be taken to reduce the density gradient in the layer because permeable material in the lower part of the layer makes it more susceptible to carbonation from below If necessary, a layer should be compacted in two parts to make the bottom less permeable.

The compaction operation should be completed within two hours and the length of road which is processed at any time should be adjusted to allow this to be achieved.

Curing. Proper curing is very important for three reasons:

- It ensures that sufficient moisture is retained in the layer so that the stabiliser can continue to hydrate.
- It reduces shrinkage.
- It reduces the risk of carbonation from the top of the layer.

In a hot and dry climate the need for good curing is very important but the prevention of moisture loss is difficult. If the surface is sprayed constantly and kept damp day and night, the moisture content in the main portion of the layer will remain stable but the operation is likely to leach stabiliser from the top portion of the layer. If the spraying operation is intermittent and the surface dries from time to time (a common occurrence when this method is used), the curing will be completely ineffective.

Spraying can be a much more efficient curing system if a layer of sand, 30 to 40 mm thick, is first spread on top of the stabilised layer. If this is done the number of spraying cycles per day can be reduced and there is a considerable saving in the amount of water used. After seven days, the sand should be brushed off and the surface primed with a low viscosity cutback bitumen.

An alternative method of curing is to first apply a very light spray of water followed by either a viscous cutback bitumen, such as MC 3000, or a slow setting emulsion. Neither of these will completely penetrate the surface of the stabilised layer and will leave a continuous bitumen film to act as a curing membrane. It is essential that all traffic is kept off the membrane for seven days. After this time, any excess bitumen can be absorbed by sanding the surface.

A prime coat cannot serve as a curing membrane. Research has shown that a prime penetrates too far into the layer and insufficient bitumen is retained on the surface to provide the necessary continuous film (Bofinger et al (1978)).

7.7.2 Control of shrinkage and reflection cracks

There is no simple method of preventing shrinkage cracks occurring in stabilised layers. However, design and construction techniques can be adopted which go some way to alleviating the problem.

Shrinkage, particularly in cement-stabilised materials, has been shown (Bofinger et al (1978)) to be influenced by

- Loss of water, particularly during the initial curing period.
- Cement content.
- Density of the compacted material.
- Method of compaction.
- Pre-treatment moisture content of the material to be stabilised.

Proper curing is essential not only for maintaining the hydration action but also to reduce volume changes within the layer. The longer the initial period of moist cure the smaller the shrinkage when the layer subsequently dries.

When the layer eventually dries, the increased strength associated with a high stabiliser content will cause the shrinkage cracks to form at increased spacing and have substantial width. With lower cement contents, the shrinkage cracks occur at reduced spacing and the material will crack more readily under traffic because of its reduced strength. The probability of these finer cracks reflecting through the surfacing is reduced, but the stabilised layer itself will be both weaker and less durable.

In order to maximise both the strength and durability of the pavement layer the material is generally compacted to the maximum density possible. However, for some stabilised materials it is sometimes difficult to achieve normal compaction standards and any increase in compactive effort to achieve them may have the adverse effect of causing shear planes in the surface of the layer or increasing the subsequent shrinkage of the material as its density is increased. If it proves difficult to achieve the target density, a higher stabiliser content should be considered in order that an adequately strong and durable layer can be produced at a lower density.

Laboratory tests have shown that samples compacted by impact loading shrink considerably more than those compacted by static loading or by kneading compaction. Where reflection cracking is likely to be a problem, it is therefore recommended that the layer should be compacted with pneumatic-tyred rollers rather than vibrating types.

Shrinkage problems in plastic gravels can be substantially reduced if air-dry gravel is used and the whole construction is completed within two hours, the water being added as late as possible during the mixing operation. It is generally not possible to use gravel in a completely air-dry condition, but the lower the initial moisture content and the quicker it is mixed and compacted, the smaller will be the subsequent shrinkage strains.

Having accepted that some shrinkage cracks are inevitable in the stabilised layer, the most effective method of preventing these from reflecting through the bituminous surfacing is to cover the cemented layer with a substantial thickness of granular material. This is the design philosophy in Charts numbered 2, 4 and 6 in Chapter 10. When cemented material is used as a roadbase (Chart 8) a flexible surfacing such as a double surface dressing is recommended. Experience in a number of countries has shown that a further surface dressing applied after 2-3 years can partially or completely seal any subsequent cracking, particularly where lime is the stabilising agent.

7.7.3 Carbonation

If cement or lime-stabilised materials are exposed to air, the hydration products may react with carbon dioxide thereby reducing the strength of the material by an average of 40 per cent of the unconfined compressive strength (Paige-Green et al (1990)). This reaction is associated with a decrease in the pH of the material from more than 12 to about 8.5. The presence and depth of carbonation can be detected by testing the pH of the stabilised layer with phenolphthalein indicator and checking for the presence of carbonates with hydrochloric acid (Netterberg (1984)). A reasonable indication of whether the material being stabilised will be subject to serious carbonation can be obtained from the wet/dry test for durability (Paige-Green et al (1990)).

Good curing practices, as outlined in Section 7.7.1, are the best means of preventing carbonation in roadbases. The risk of carbonation can be reduced by taking the following precautions:

- Avoid wet/dry cycles during the curing phase.
- Seal as soon as possible to exclude carbon dioxide.
- Compact as early as possible to increase the density and to reduce the permeability.
- Reduce the possibility of reflection cracks.

There may be some conflict between the last two points and care should be taken not to over compact the layer.

Checks should be made during construction and if the depth of carbonated material is more than 2 to 3 mm the carbonated layer should be removed by heavy brushing or grading before the surfacing is applied.

7.8 QUALITY CONTROL

A high level of quality control is necessary in the manufacture of cement and lime-stabilised materials, as with all other materials used in the road pavement, but several factors need special consideration.

***Storage and handling of stabilisers**.* Unless cement and lime are properly stored and used in a fresh condition the quality of the pavement layer will be substantially reduced. Cement must be stored in a solid, watertight shed and the bags stacked as tightly as possible. Doors and windows should only be opened if absolutely necessary. The cement which is delivered from the manufacturer first should also be used first. Even if cement is properly stored the following losses in strength will occur:-

After 3 months	20% reduction
After 6 months	30% reduction
After 1 year	40% reduction
After 2 years	50% reduction

Lime should be packed in sealed bags, tightly stacked and stored under cover or at least under a watertight tarpaulin. If it becomes contaminated or damp, it can only be used as a filler. Lime which is older than 6 months should be discarded.

Distribution of stabiliser. After the layer has been properly processed, at least 20 samples should be taken for determination of the stabiliser content. The mixing efficiency is acceptable if the coefficient of variation is less than 30 per cent. Great care is necessary in multi-layer construction to ensure that good mixing extends to the full depth of all the layers.

Opening to traffic**.* Insufficient research has been carried out to determine the precise effects of opening a road to traffic before the completion of the curing period but it is considered that allowing traffic on the pavement during the first two days can be beneficial for some stabilised layers provided the traffic does not mark the 'green' surface and ***all traffic is kept off the pavement from the end of the second day until one week has elapsed (Williams (1986)). Early trafficking has a similar effect to that of pre-cracking the layer by rolling within a day or two of its construction but rolling is preferred because it ensures even coverage of the full width of the carriageway.

Layers which are pre-cracked or trafficked early must be allowed to develop sufficient strength to prevent abrasion of the edges of each crack before the layer is opened to general traffic. The slab strength of these layers is effectively destroyed and it is recommended that early trafficking is only acceptable for layers of cemented roadbase type CB2.

8. BITUMEN-BOUND MATERIALS

This chapter describes types of bituminous materials, commonly referred to as premixes, which are manufactured in asphalt mixing plants and laid hot. In situ mixing using either labour intensive techniques or mechanised plant can also be used for making roadbases for lower standard roads but these methods are not generally recommended and are not discussed in detail here.

8.1 COMPONENTS OF A MIX

The coarse aggregates used for making premix should be produced by crushing sound, unweathered rock or natural gravel. The specifications for the aggregates are similar to those for granular roadbases. The aggregate must be clean and free of clay and organic material. To obtain good mechanical interlock and good compaction the particles should be angular and not flaky. Rough-textured material is preferable. Gravel should be crushed to produce at least two fractured faces on each particle. The aggregate must be strong enough to resist crushing during mixing and laying as well as in service. Aggregates which are exposed to traffic must also be resistant to abrasion and polishing. Highly absorptive aggregates are wasteful of bitumen and give rise to problems in mix design. They should be avoided where possible but if there is no choice, the absorption of bitumen must be taken into account in the mix design procedure. Hydrophillic 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 per cent 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.

8.2 BITUMINOUS SURFACINGS

The most critical layer of the pavement is the bituminous surfacing, and the highest quality material is necessary for this layer. Where thick bituminous surfacings are required, they are normally constructed with a wearing course laid on a basecourse (sometimes called a binder course) which can be made to slightly less stringent specifications.

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

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

The requirements of a mix which will ensure each of these characteristics are often conflicting In temperate climates it has proved possible to design mixes which possess an acceptable balance of properties giving long service lives under a range of loading and climatic conditions. In the tropics, higher temperatures and high axle loads produce an environment which is more severe thereby making the mix requirements more critical and an overall balance of properties more difficult to obtain.

High temperatures initially reduce the stiffness of mixes, making them more prone to deformation, and also cause the bitumen to oxidise and harden more rapidly, thereby reducing its durability. Unfortunately the requirements for improved durability i e. increased bitumen content and lower voids, usually conflict with the requirements for higher stiffness and improved deformation resistance. As a result, the tolerances on mix specifications need to be very narrow and a high level of quality control at all stages of manufacture is essential. The requirements are so critical for wearing course mixes that different mix designs are often necessary for different conditions on the same road. For example, mixes suitable for areas carrying heavy, slow-moving traffic, such as on climbing lanes, or areas where traffic is highly channelled, 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 (Hoban (1990), M. Hizam Harun and Jones (1992)).

The age hardening of the bitumen in the wearing course is much greater at the exposed surface where the effect of the environment is much more severe and it is this hardened, brittle skin that usually cracks early in the life of the surfacing (Rolt et al (1986)). In areas where the diurnal temperature range is large, for example in most desert areas, thermal stresses can significantly increase the rate at which cracking occurs. The risk of premature cracking can be greatly reduced by applying a surface dressing to the wearing course soon after it has been laid, preferably after a few weeks of trafficking by construction traffic. This provides a bitumen-rich layer with a high strain tolerance at the point of potential weakness whilst also providing a good surface texture with

TABLE 8.1

Coarse aggregate for bituminous mixes

Property	Test	Specification
Cleanliness	Sedimentation or Decantation (1,2)	< 5 per cent passing 0.075 mm sieve
Particle shape	Flakiness Index (3)	< 45 per cent
Strength	Aggregate Crushing Value (ACV) (4)	< 25. For weaker aggregates the Ten per Cent Fines Value Test (TFV) is used.
	Aggregate Impact Value (AIV) (4)	< 25
	Los Angeles Abrasion Value (LAA) (5)	< 30 (wearing course) < 35 (other)
Abrasion	Aggregate Abrasion Value (AAV) (4)	< 15 < 12 (very heavy traffic)
Polishing (wearing course only)	Polished Stone Value (4)	Not less than 50-75 depending on location
Durability	Soundness: (6) Sodium Test Magnesium Test	 < 12 per cent < 18 per cent
Water Absorption	Water Absorption (7)	< 2 per cent
Bitumen Affinity	Immersion Tray Test (8) Effect of water on cohesion of compacted mixes	 Index of retained stability > 75 per cent

Notes.
1. British Standard 812, Part 103 (1985)
2. J C Bullas and G West (1991)
3. British Standard 812, Part 105 (1990)
4. British Standard 812, Part 3 (1985)
5. ASTM C131 and C535
6. British Standard 812, Part 121 (1989)
7. British Standard 812, Part 2 (1975)
8. Shell Bitumen Handbook, D Whiteoak (1990)

improved skid resistant properties. If such a surface dressing is used, some cost savings can often be made by using a basecourse material in place of the wearing course. For severely loaded sites, such mixes can be designed to have a high resistance to deformation and under these conditions a surface dressing is essential if early cracking is to be prevented.

It has also been shown (Smith et al (1990)) 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 pen 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, D2872, Dickinson (1982)).

8.3 TYPES OF PREMIX IN COMMON USE

The main types of premix are asphaltic concrete, bitumen macadam and hot rolled asphalt. Each type can be used in surfacings or roadbases. Their general properties and specifications suitable for tropical environments are described below. A design procedure based on 'refusal density' is suggested to enhance the standard Marshall procedure (Section 8.3 5 and Appendix D).

TABLE 8.2

Fine aggregate for bituminous mixes

Property	Test	Specification
Cleanliness	Sedimentation or Decantation (1,2)	Per cent passing 0.075 mm sieve:- Wearing courses < 8 per cent for sand fines < 17 per cent for crushed rock fines Other layers: < 22 per cent
	Sand Equivalent (material passing 4.75 mm sieve)	Traffic / Wearing Course / Basecourse Light (<T3) / > 35% / > 45% Medium/Heavy / > 40% / > 50%
	Plasticity Index (material passing 0.425 mm sieve)	< 4
Durability	Soundness Test (6) (5 cycles)	Magnesium : < 20 per cent Sodium : < 15 per cent

See Notes to Table 8.1.

8.3.1 Asphaltic concrete

Asphaltic concrete (AC) is a dense, continuously graded mix which relies for its strength on both the interlock between aggregate particles and, to a lesser extent, on the properties of the bitumen and filler. The mix is designed to have low air voids and low permeability to provide good durability and good fatigue behaviour but this makes the material particularly sensitive to errors in proportioning, and mix tolerances are therefore very narrow (Jackson and Brien (1962), Asphalt Institute (1983), (1989) and (1991)).

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 but they are not ideal for conditions of severe loading e.g. slow moving heavy traffic and high temperatures (see Section 8.3.5). This is because the continuous matrix of fine aggregate, filler and bitumen is more than sufficient to fill the voids in the coarse aggregate and this reduces the particle to particle contact within the coarse aggregate and lowers the resistance to deformation. A particle size distribution that conforms to the requirements for asphaltic concrete or a close graded bitumen macadam basecourse (BC1 in Table 8.3 or BC2 in Table 8.6) is recommended for use as the wearing course in severe conditions but such mixes must be sealed.

It is common practice to design the mix using the Marshall Test and to select the design binder content by calculating the mean value of the binder contents for (i) maximum stability, (ii) maximum density, (iii) 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).

A maximum air voids content of 5 per cent is recommended to reduce the potential age hardening of the bitumen but on severe sites the overriding criteria is that a minimum air voids of 3 per cent at refusal density should be achieved. This is equivalent to the condition which will arise after heavy trafficking and is designed to ensure that serious deformation does not occur. For such a mix it is unlikely that it will also be possible to reduce the air voids content at 98 per cent of Marshall density to 5 per cent and therefore it is recommended that a surface dressing is applied to the wearing course to provide the necessary protection against age hardening.

It is frequently found that mixes are designed to have the highest possible stabilities. This usually means that the binder content is reduced resulting in mixes which are more difficult to compact and are less durable. It is important to note that there is a relatively poor correlation between Marshall stability and deformation in service, and durability should not be jeopardised in the belief that a more deformation resistant mix will be produced.

A better 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 centre of the common range. If this common range is too narrow, the aggregate

TABLE 8.3

Asphaltic concrete surfacings

Mix designation	WC1	WC2	BC1
	Wearing course		*Basecourse*
BS test sieve (mm)	Percentage by mass of total aggregate passing test sieve		
28	-	-	100
20	100	-	80 - 100
14	80 - 100	100	60 - 80
5	54 - 72	62 - 80	36 - 56
2.36	42 - 58	44 - 60	28 - 44
1.18	34 - 48	36 - 50	20 - 34
0.6	26 - 38	28 - 40	15 - 27
0.3	18 - 28	20 - 30	10 - 20
0.15	12 - 20	12 - 20	5 - 13
0.075	6 - 12	6 - 12	2 - 6
Bitumen content (1) (per cent by mass of total mix)	5.0 - 7.0	5.5 - 7.4	4.8 - 6.1
Bitumen grade (pen)	60/70 or 80/100	60/70 or 80/100	60/70 or 80/100
Thickness (2) (mm)	40 - 50	30 - 40	50 - 65

Notes. 1. Determined by Marshall design method
2. In practice the upper limit has been exceeded by 20 per cent with no adverse effect.

TABLE 8.4

Suggested Marshall Test criteria

Total Traffic (10^6 esa)	< 1.5	1.5 - 10.0	> 10.0	Severe sites(1)
Traffic classes	T1,T2,T3	T4,T5,T6	T7,T8	-
Minimum stability (kN at 60°C)	3.5	6.0	7.0	9.0
Minimum flow (mm)	2	2	2	2
Compaction level (Number of blows)	2 x 50	2 x 75	2 x 75	2 x 75
Air voids (per cent)	3 - 5	3 - 5	3 - 5	3 - 5(2)

Notes. 1. Slow moving heavy traffic, etc. (see Section 8.3.4)
2. The refusal density must be satisfactory

grading should be adjusted until the range is wider and tolerances less critical.

To ensure that the compacted mineral aggregate in continuously graded mixes has a voids content large enough to contain sufficient bitumen, a minimum value of the voids in the mineral aggregate (VMA) is specified, as shown in Table 8.5.

TABLE 8.5

Voids in the mineral aggregate

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

The Marshall design procedure is based on the assumption that the densities achieved in the Marshall Test samples represent those that will occur in the wheelpaths after a few years of trafficking. If in situ air voids are too high, rapid age hardening of the bitumen will ensue. Conversely, on severely loaded sites the air voids may be reduced by traffic leading to failure through plastic flow. In this latter situation the method of designing for a minimum air voids in the mix (VIM) at refusal density should be used (see Section 8 3.5).

8.3.2 Bitumen macadam

Close graded bitumen macadams (formerly called dense bitumen macadams or DBMs) are continuously graded mixes similar to asphaltic concretes but usually with a less dense aggregate structure. They have been developed in the United Kingdom (British Standard 4987 (1984)) from empirical studies over many years and are made to recipe specifications without reference to a formal design procedure. Implicit in the design is a knowledge of which materials perform satisfactorily in the United Kingdom under given climatic conditions and strictly controlled vehicle axle loads. Doubts about their suitability for different conditions and with different materials may therefore arise 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 both wearing course and basecourse mixes 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.

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. Recipe mixes are not recommended in these circumstances and 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.

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.

Rolled asphalt has been developed in the United Kingdom 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 (1985)). 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 summarised in Table 8 8. 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.

8.3.4 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 catalogue are flexible. This is particularly important for surfacings laid on granular roadbases. 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 ageing and embrittlement of the bitumen must be prevented by applying a surface dressing.

8.3.5 Design to refusal density

Under severe loading conditions asphalt mixes must be expected to experience significant secondary compaction in the wheelpaths. Severe conditions cannot be precisely

TABLE 8.6

Bitumen macadam surfacings

Mix designation	WC3	WC4	BC2
	Wearing Course (5)		*Basecourse*
BS test sieve (mm)	Percentage by mass of total aggregate passing test sieve		
28	-	-	100
20	100	-	95 - 100
14	95 - 100	100	65 - 85
10	70 - 90	95 - 100	52 - 72
6.3	45 - 65	55 - 75	39 - 55
3.35	30 - 45	30 - 45	32 - 46
1.18	15 - 30	15 - 30	-
0.3	-	-	7 - 21
0.075 (1)	3 - 7	3 - 7	2 - 8
Bitumen grade (2) (pen)	80/100 or 60/70	80/100 or 60/70	80/100 or 60/70
Bitumen content (3) (per cent by mass of total mix)	5.3 ± 0.5	5.5 ± 0.5	5.0 ± 0.6
Thickness (4) (mm)	40 - 55	30 - 40	50 - 80

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

TABLE 8.7

Suggested Marshall criteria for close graded bitumen macadams

Design traffic (10^6 esa)	< 1.5	1.5 - 10	>10	Severe sites
Traffic classes	T1,T2,T3	T4,T5,T6	T7,T8	-
Minimum stability (kN at 60°C)	3.5	6.0	7.0	9.0
Flow (mm)	2-4	2-4	2-4	2-4
Compaction level (number of blows)	2 x 50	2 x 75	To refusal	To refusal

TABLE 8.8

Rolled asphalt surfacings

Mix designation	WC5	WC6	BC3	BC4
	Wearing Course [1,2]		*Basecourse*	
BS test sieve (mm)	Percentage by mass of total aggregate passing test sieve			
28	-	-	-	100
20	100	100	100	90 - 100
14	90 - 100	90 - 100	90 - 100	65 - 100
10	50 - 85	50 - 85	65 - 100	35 - 75
2.36	50 - 62	50 - 62	35 - 55	35 - 55
0.6	35 - 62	20 - 40	15 - 35	15 - 55
0.212	10 - 40	10 - 25	5 - 30	5 - 30
0.075	6 - 10	6 - 10	2 - 9	2 - 9
Type of fines	Natural sand	Crushed rock	Sand or crushed rock	
Bitumen grade (pen)	40/50 or 60/70	60/70	40/50 or 60/70	
Thickness (mm)	50	50	40-65	50-80
Bitumen content (per cent by mass of total mix)	Minimum target value 6.3 ± 0.5 [3]		6.5 ± 0.6 (crushed rock) 6.3 ± 0.6 (gravel)	

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 per cent coarse aggregate (see BS594).

defined but will consist of a combination of two or more of the following;

- High maximum temperatures
- Very heavy axle loads
- Very channelled 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.

8.4 BITUMINOUS ROADBASES

Satisfactory bituminous roadbases for use in tropical environments can be made using a variety of specifications. They need to 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 roadbases in the tropics are higher than in temperate climates and the mixes are therefore more prone to deformation in early life, and ageing and embattlement later.

8.4.1 Principal mix types

Particle size distributions and general specifications for continuously graded mixes are given in Table 8.9. 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. Suitable specifications for gap-graded rolled asphalt roadbases are given in Table 8.10.

All these specifications are recipes which have been developed from experience and rely on performance data.

TABLE 8.9

Bitumen macadam roadbase

Mix designation	RB1
BS test sieve (mm)	Percentage by mass of total aggregate passing test sieve
50	100
37.5	95 - 100
28	70 - 94
14	56 - 76
6.3	44 - 60
3.35	32 - 46
0.3	7 - 21
0.075	2 - 8[1]
Bitumen content (per cent by mass of total mix)	4.0[2] ± 0.5
Thickness (mm)	65 - 125
Voids (per cent)	4 - 8
Bitumen grade (pen)	60/70 or 80/100

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

TABLE 8.10

Rolled asphalt roadbase

Mix designation	RB2	RB3
BS test sieve (mm)	Percentage by mass of total aggregate passing test sieve	
50	-	100
37.5	100	90 - 100
28	90 - 100	70 - 100
20	50 - 80	45 - 75
14	30 - 60	30 - 65
2.36	30 - 44	30 - 44
0.6	10 - 44	10 - 44
0.212	3 - 25	3 - 25
0.075	3 - 7	3 - 7
Bitumen content (per cent by mass of total mix)	5.7 ± 0.6	
Layer thickness (mm)	60 - 120	75 - 150
Filler:binder ratio	0.6 - 1.2	
Bitumen grade (pen)	40/50 or 60/70	

for their optimum adaptation to local conditions. The following principles should be adopted for all bituminous layers but are particularly important for recipe type specifications:

(i) Trials for mix production, laying and compaction should be carried out to determine suitable mix proportions and procedures.

(ii) Durable mixes require a high degree of compaction and this is best achieved by specifying density in terms of maximum theoretical density of the mix or, preferably, by using a modification of the Percentage Refusal Test with extended compaction time. (British Standard 598, Part 104 (1989), Powell and Leech (1982)).

(iii) Mixing times and temperatures should be set at the minimum required to achieve good coating of the aggregates and satisfactory compaction.

(iv) The highest bitumen content commensurate with adequate stability should be used.

8.4.2 Sand-bitumen mixes

For light and medium trafficked roads (defined as roads carrying less than 300 commercial vehicles per day and with mean equivalent standard axles per vehicle of 0 5 or less) and in areas lacking coarse aggregates, bitumen-stabilised sands are an alternative. Best results are achieved with well graded angular sands in which the proportion of material passing the 0 075 mm sieve does not exceed ten per cent 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 use of penetration grade bitumens will produce the highest stabilities but this will necessitate heating the sand as well as the bitumen. An example has been given by Harris et al (1983).

The amount of bitumen required will generally he 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 can be used for determining the amount of bitumen required (Asphalt Institute, MS-2 (1988)) Design criteria are given in Table 8.11 for sand-bitumen mixes used as roadbase materials for tropical roads carrying medium to light traffic.

TABLE 8.11

Criteria for sand-bitumen roadbase materials

	Traffic Classes	
	T1	T2
Marshall stability at 60°C (min)	1 kN	1.5 kN
Marshall flow value at 60°C (max)	2.5 mm	2 mm

8.5 MANUFACTURE AND CONSTRUCTION

General guidance on the design, manufacture and testing of bitumen macadams and rolled asphalts can be found in the British Standards, BS 4987 (1988) for macadams and BS 594 (1985) and BS 598 (1985) for rolled asphalts. Similar guidance for asphalt concrete is given in the publications of the Asphalt Institute, SS-1 (1980), MS-2 (1988) and MS-22 (1983), and the US Army Corps of Engineers (1991).

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 job-mix particle size distribution should be reasonably parallel to the specified grading envelope and is the target grading for the mix to be produced by the asphalt plant. Loss of fines may occur during the drying and heating phase and, therefore, tests on aggregates which have passed through the asphalt plant in the normal way should be used to establish a job-mix formula which meets the specified Marshall Test criteria.

The importance of detailed compaction trials at the beginning of asphalt construction work cannot be over emphasised. 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 12. 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 13. Very little additional compaction is achieved at the minimum rolling temperatures shown in the Table and only pneumatic tyred rollers should be used at these temperatures.

Rolled asphalts are relatively easy to compact but bitumen macadams and asphaltic concretes are relatively harsh and more compactrve effort is required. Heavy pneumatic tyred rollers are usually employed, the

TABLE 8.12

Job-mix tolerances for a single test

Combined aggregate passing test sieve (mm)		Bitumen content	
BS test sieve	Per cent	Mix type	Per cent
12.5+	± 5		
10.0	± 5	Wearing courses	± 0.3
2.36	± 5		
0.60	± 4		
0.30	± 3	Basecourses	± 0.4
0.15	± 2		
0.075	± 2	Roadbases	± 0.4

TABLE 8.13

Manufacturing and rolling temperatures (°C)

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

kneading action of the tyres 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 tyred 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 centre 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 minimised by using two pavers in echelon or 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 Asphalt Institute method of joint formation is recommended. The edge of the first layer must be `rolled 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 material will allow a satisfactory joint to be constructed.

9. SURFACE TREATMENTS

9.1 PRIME AND TACK COATS

A prime coat is a thin layer of bitumen sprayed onto the surface of an existing layer, usually of unbound or cement/lime bound material. Its purpose can be summarised as follows:

- It assists in promoting and maintaining adhesion between the roadbase and the bituminous surfacing by pre-coating the surface of the roadbase and by penetrating the voids near the surface.

- It helps to seal the surface pores in the roadbase, thus reducing the absorption of the first spray of bitumen of a surface dressing.

- It helps to bind the finer particles of aggregate together in the surface of the roadbase.

- If the application of the surfacing is delayed for some reason, it provides the roadbase with temporary protection against the detrimental effects of rainfall and light traffic.

Low viscosity, medium curing cutback bitumens such as MC-30, MC-70, or in rare circumstances MC-250, can be used for prime coats (alternatively low viscosity road tar can be used if this is available). The depth of penetration should be about 3-10 mm and the quantity sprayed should be such that the surface is dry within two days. The correct viscosity and application rate are dependent primarily on the texture and density of the surface being primed. The application rate is likely to lie within the range 0.3-1 1 kg/m2.

Low viscosity cutbacks are necessary for very dense cement or lime-stabilised surfaces, and high viscosity cutbacks for untreated coarse-textured surfaces. It is .usually helpful 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 because they tend to form a skin on the surface.

The primary function of a tack coat is to act as a glue to assist bonding of a new surface layer to a previously primed surface, bituminous roadbase, or basecourse that has been left exposed for some time. Tack coats should be extremely thin and it is appropriate to use a dilute bitumen emulsion spread to give less than 0.2 kg /M^2 of residual bitumen with continuous cover. When temperature conditions are satisfactory, it is possible to obtain a thin layer by lightly spraying the undiluted emulsion with a hand-lance and then spreading it with a pneumatic tyred roller to obtain complete coverage.

9.2 SURFACE DRESSING

The design of surface dressing is described in detail in Overseas Road Note 3 (TRRL (1982)). The design should take into account the type of existing road surface, the traffic, the available chippings and the climate.

9.2.1 Single and double surface dressings

Single surface dressings are normally adequate when applied to a bituminous layer. To be satisfactory for non-bituminous surfacings, the quality of a single seal must be very high and subsequent minor maintenance must be carried out promptly when required.

It is recommended that double surface dressings are always used on non-bituminous layers. The quality of a double surface dressing will be greatly enhanced if traffic is allowed to run on the first dressing for a minimum period of 2-3 weeks (and preferably longer) before the second dressing is applied. This allows the chippings of the first dressing to adopt a stable interlocking mosaic which provides a firm foundation for the second dressing. If the trafficking results in the contamination of the first dressing with mud or soil, this should be thoroughly cleaned off before the second dressing is applied.

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

A surface dressing applied to the shoulders of paved roads can be an effective method of improving drainage and reducing erosion and pavement edge damage. There are several factors which need to be considered, their importance being dependent on traffic intensity.

- The shoulder material must be suitable for surface dressing and be strong enough to support stationary, heavy wheel loads.

- Effective 'overbanding' is necessary to give a heavy duty transition seal between the shoulder and pavement surface.

- Shoulder maintenance by motor grader will not be possible. Age hardening of the shoulder seal will mean that maintenance reseals will be required at approximately five yearly intervals.

- Edge markings or the use of different coloured aggregate will be necessary to delineate the edge of the running surface.

- On uphill grades it may be necessary to use rumble strips on the shoulders to prevent them being used for overtaking.

9.2.2 Type of surface

Embedment of the chippings under traffic is dependent upon the hardness of the layer to be sealed and the size of the chippings. Assessment of layer hardness can be

TABLE 9.1

Categories of road surface hardness

Surface category	Penetration * at 30°C (mm)	Definition
Very hard	0 - 2	Surfaces such as concrete or chemically stabilised roadbases into which negligible penetration of chippings will occur under heavy traffic
Hard	2 - 5	Granular roadbases into which chippings will penetrate only slightly under heavy traffic
Normal	5 - 8	Bituminous roadbases or basecourses into which chippings will penetrate moderately under medium and heavy traffic
Soft	8 - 12	Bitumen rich asphalts into which chippings will penetrate considerably under medium and heavy traffic

*See Appendix E.

based on descriptive definitions or measured using a simple penetration test probe. Details of surface category, penetration values, and descriptive definitions are given in Table 9.1. The probe penetration test is described in Appendix E.

9.2.3 Traffic categories

The volume of traffic is considered in terms of the number of commercial vehicles per day in the lane under consideration. The traffic categories are defined in Table 9 2. It should be noted that these differ from the traffic classes used in the selection of the pavement structure in Chapter 10.

TABLE 9.2

Traffic categories for surface dressing

Category	Approximate number of vehicles with unladen weight greater than 1.5 tonnes (per day)
1	Over 2000
2	1000 - 2000
3	200 - 1000
4	20 - 200
5	Less than 20

9.2.4 Chippings

The chippings should comply in all respects with the requirements in British Standards 63, Part 2 (1987). The size of chippings should be chosen to suit the level of traffic and the hardness of the underlying surface as shown in Table 9.3.

In selecting the nominal size of chippings for double surface dressings, 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 3. The nominal size of chipping selected for the

TABLE 9.3

Recommended maximum chipping size (mm)

Surface category	Traffic category 1	2	3	4	5
Very hard	10	10	6	6	6
Hard	14	14	10	6	6
Normal	20	14	14	10	6
Soft	*	20	14	14	10

* Not suitable for surface dressing

second layer should then be about half the nominal size of the first layer to promote good interlock between the layers a g. a 20 mm first layer should be followed by a 10 mm second layer, or a 14 mm first layer should be followed by a 10 mm or 6 mm second layer.

In the case of a hard existing surface where little embedment of the first layer of chippings is possible, such as a newly constructed cement-stabilised roadbase or a dense crushed rock roadbase, a 'pad coat' of 6 mm chippings should be applied first followed by 10 mm or 14 mm 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.

The quantity of chippings must be sufficient to cover the entire surface of the binder film after rolling. The rate at which chippings should be spread depends on their size, shape and specific gravity, but rates can be estimated using Figure 7.

The least dimension of at least 200 chippings should be measured and the 'Average Least Dimension' (ALD) determined. An alternative method based on median particle size and Flakiness Index is described in Overseas Road Note 3 (TRRL 1982)). The ALD is then used in the Figure together with the line labelled AB and the approximate rate of application of chippings read from the upper scale. This rate should be used as a guide for supply purposes. The actual rate of spread should be adjusted on site when the spreading characteristics of the chippings have been observed.

9.2.5 Binder

Figure 8 shows the viscosity/road temperature relationships for a wide range of binders. In the tropics, day-time road temperatures range from 20'C to 70°C. The Figure indicates that the most appropriate binders are likely to be MC 3000 or the penetration grades up to 80/100. If the correct binder is not available it is sometimes possible to blend suitable materials on site (Hitch and Stewart (1987)).

To determine the rate of application of binder, an appropriate factor should be selected from Table 9.4 for each of the four sets of conditions listed. The four factors are then added together to give the total weighting factor. The Average Least Dimension of the chippings and the total weighting factor obtained from the condition constants in Table 9.4 are then used with Figure 7 to obtain the rate of application of binder. Research in Kenya (Hitch, (1981)) has shown that the rate of spread of binder should be adjusted to take account of road gradient and traffic speed as indicated in the Figure.

Finally the rate of spread of binder needs to be modified to allow for the different proportions of residual bitumen in the different binders. No adjustment is needed for MC3000 but for penetration grade bitumens the spray rate should be reduced by 10 per cent for 80/100 penetration grade and 5 per cent for 300 penetration grade. For emulsions it should be increased by the factor (90/ bitumen content of the emulsion in per cent).

9.3 SLURRY SEALS

Slurry seals are mixtures of fine aggregates, Portland cement filler, bitumen emulsions and additional water (ASTM D3910-84 (1990); BS 434, Parts 1 and 2, (1984)). When freshly mixed, they have a thick 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 surface dressing, it does not provide good surface texture, and it is considerably less durable.

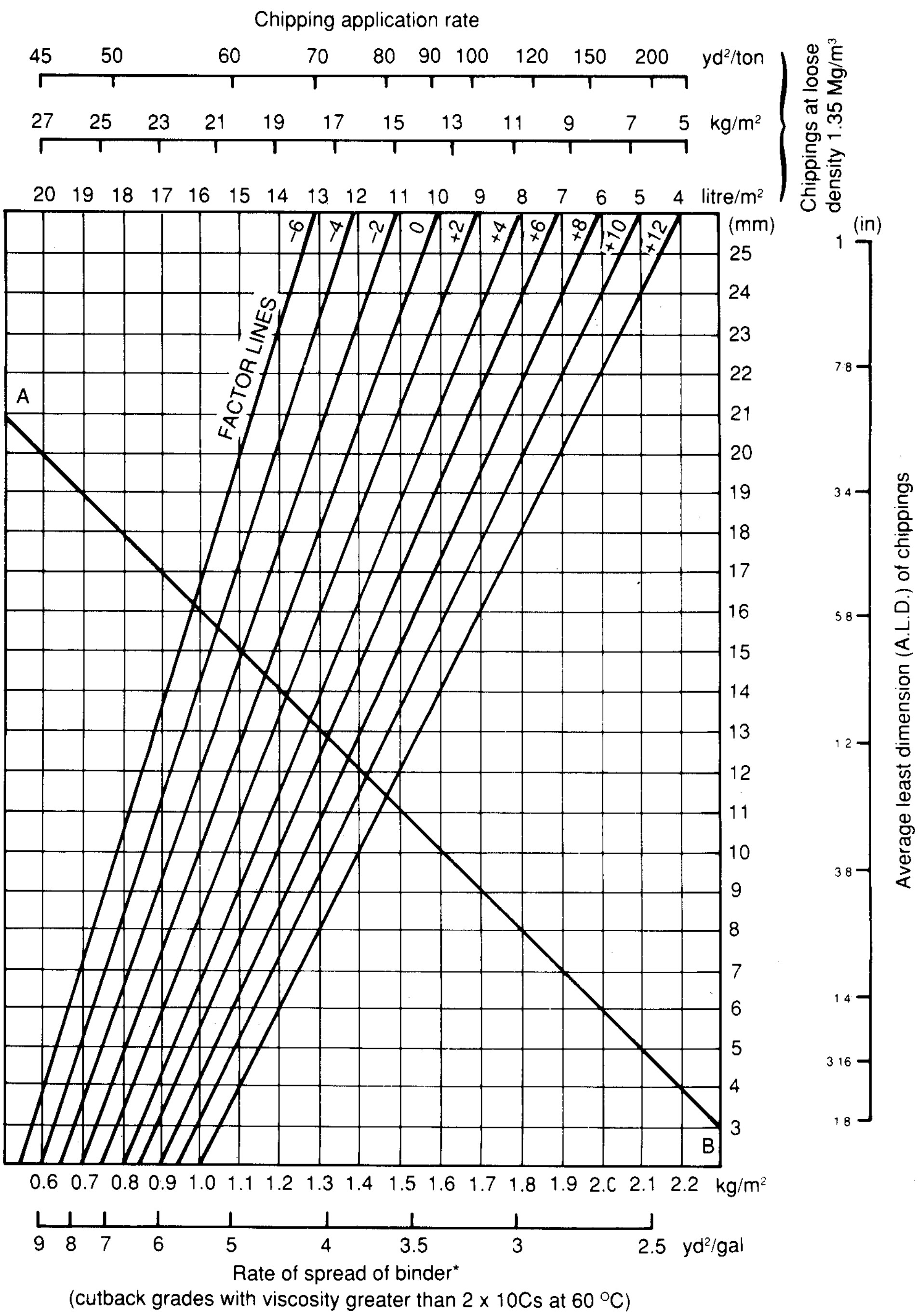
TABLE 9.4

Condition constants for determining the rate of application of binder

Traffic	Vehicles/day*	Constant	Type of Chippings	Constant
Very light	0 - 50	+3	Round/dusty	+2
Light	50 - 250	+1		
Medium	250 - 500	0	Cubical	0
Medium Heavy	500 - 1500	–1		
Heavy	1500 - 3000	–3	Flaky	–2
Very Heavy	3000+	–5		
			Pre-coated	–2

Existing surface	Constant	Climatic conditions	Constant
Untreated/primed roadbase	+6	Wet and cold	+2
Very lean bituminous	+4	Tropical (wet and hot)	+1
Lean bituminous	0	Temperate	0
Average bituminous	–1	Semi-arid (dry and hot)	–1
Very rich bituminous	–3	Arid (very dry and very hot)	–2

*All vehicles in one direction



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

Fig.7 Surface dressing design chart

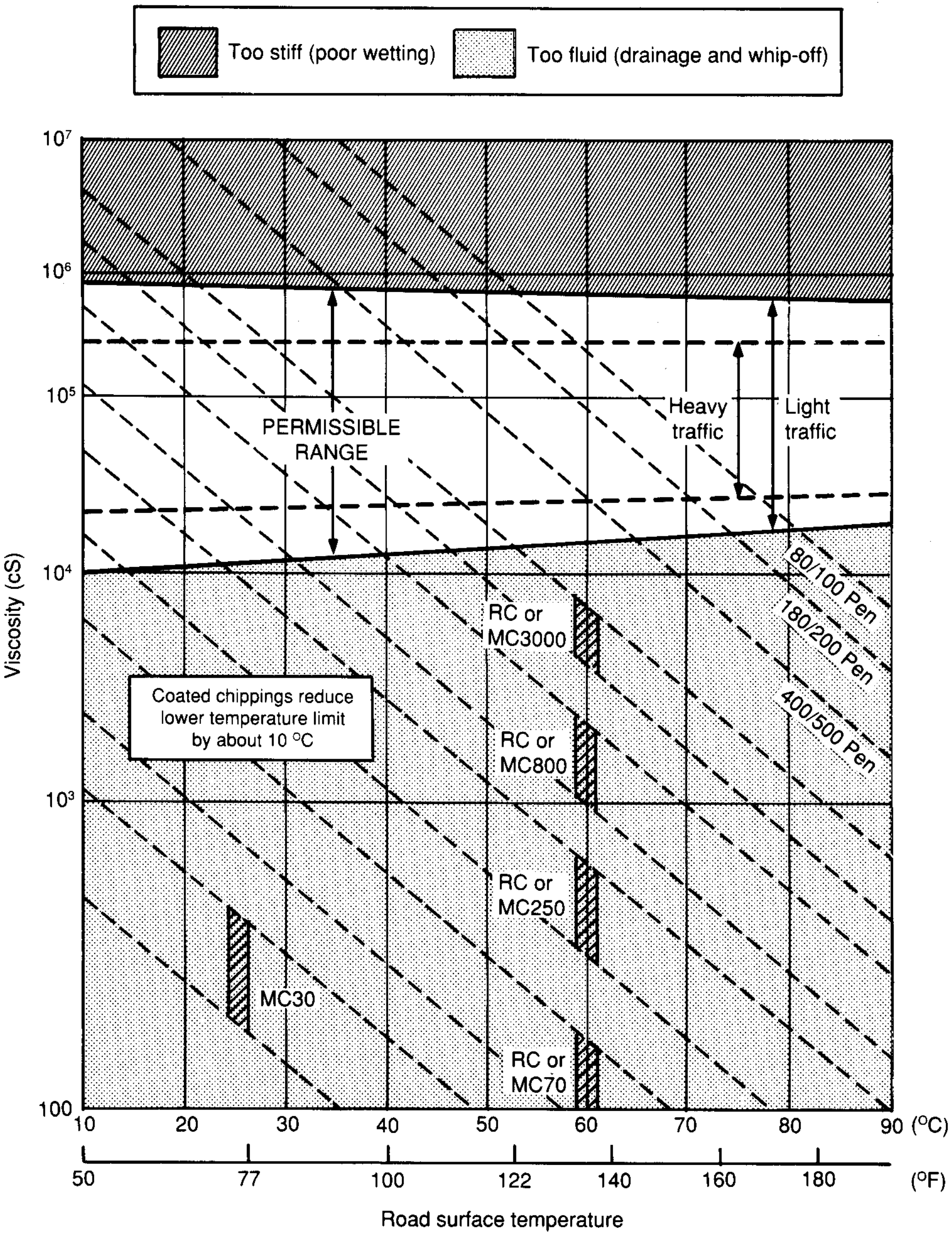


Fig.8 Surface temperature and choice of binder for surface dressing

Slurry seals are often used in combination with a surface dressing to make a 'Cape seal'. In this technique the slurry seal is applied on top of a single surface dressing to produce a surface texture which is less harsh than a surface dressing alone and a surface which is flexible and durable. However, the combination is more expensive than a double surface dressing 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 advantageous when rainfall is likely to occur. A suitable specification for slurry seals is given in Table 9.5.

TABLE 9.5

Specifications for slurry seals

Particle size distribution

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

Note. The optimum mix design for the aggregate, filler, water and bitumen emulsion mixture should be determined using ASTM D 3910-84 (1990).

Coverage

Surface aggregate	Coverage (m^2/m^3)
20 mm	130 - 170
14 mm	170 - 240
10 mm	180 - 250
Primed roadbase	150-180 (in 2 layers)

10. STRUCTURE CATALOGUE

The basis of the catalogue has been described in Section 1.7 and most of the information necessary to use it is contained in the main chapters of this Road Note. The cells of the catalogue are defined by ranges of traffic (Chapter 2) and subgrade strength (Chapter 3) and all the materials are described in Chapters 6 to 9. A summary of requirements and reference chapters relevant to each design chart is given in Table 10 1.

Although the thicknesses of layers should follow the designs whenever possible, some limited substitution of materials between sub-base and selected fill is allowable based on the structural number principles outlined in the AASHTO guide for design of pavement structures (AASHTO (1986)). Where substitution is allowed, a note is included with the design chart.

The charts are designed so that, wherever possible, the thickness of each lift of material is obvious. Thus, all layers less than 200 mm will normally be constructed in one lift and all layers thicker than 300 mm will be constructed in two lifts. Occasionally layers are of intermediate thickness and the decision on lift thickness will depend on the construction plant available and the ease with which the density in the lower levels of the lift can be achieved. The thickness of each lift need not necessarily be identical and it is often better to adjust the thickness according to the total thickness required and the maximum particle size by using a combination of gradings from Table 6.2.

In Charts 3, 4 and 7 where a semi-structural surface is defined, it is important that the surfacing material should be flexible (Chapter 8) and the granular roadbase should be of the highest quality, preferably GB1,A. In traffic classes T6, T7 and T8 only granular roadbases of type GB1 or GB2 should be used, GB3 is acceptable in the lower traffic classes. For lime or cement-stabilised materials, the charts already define the layers for which the three categories of material may be used.

The choice of chart will depend on a variety of factors but should be based on minimising total transport costs as

TABLE 10.1

Summary of material requirements for the design charts

CHART NO	SURFACING	ROADBASE	REFER TO CHAPTERS
1	Double surface dressing	T1-T4 use GB1,GB2 or GB3 T5 use GB1,A or GB1,B T6 must be GB1,A	6 and 9
2	Double surface dressing	T1-T4 use GB1, GB2 or GB3 T5 use GB1 T6,T7,T8 use GB1,A	6, 7 and 8
3	'Flexible' asphalt	T1-T4 use GB1 or GB2 T5 use GB1 T6 use GB1,A	6 and 8
4	'Flexible' asphalt	T1-T4 use GB1 or GB2 T5 use GB1 T6-T8 use GB1,A	6, 7 and 8
5	Wearing course and basecourse	GB1,A	6 and 8
6	Wearing course and basecourse	GB1 or GB2	6, 7 and 8
7	High quality single seal or double seal for T4. 'Flexible' asphalt for T5-T8	RB1, RB2 or RB3	8 and 9
8	Double surface dressing	CB1, CB2	7 and 9

discussed in Section 1 3 Factors that will need to be taken into account in a full evaluation include,

- the likely level and timing of maintenance
- the probable behaviour of the structure
- the experience and skill of the contractors and the availability of suitable plant
- 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, as described in the text, and therefore the initial choice should be based on the local costs of the feasible options If any information is available concerning the likely behaviour of the structures under the local conditions, then a simple risk analysis can also be carried out to select the most appropriate structure (e g Ellis (1975)). With more detailed information, it should be possible to calibrate one of the road investment models such as HDM-111 (Watanatada et al (1987)) or RTIM-2 (Parsley and Robinson (1982)) and then to use the model to calculate the whole life costs associated with each of the possible structures thereby allowing the optimum choice to be made. 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.

KEY TO STRUCTURAL CATALOGUE

Traffic classes
(10^6 esa)

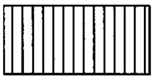
T1 = < 0.3
T2 = 0.3 - 0.7
T3 = 0.7 - 1.5
T4 = 1.5 - 3.0
T5 = 3.0 - 6.0
T6 = 6.0 - 10
T7 = 10 - 17
T8 = 17 - 30

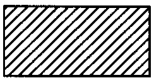
Subgrade strength classes
(CBR%)

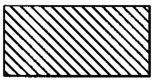
S1 = 2
S2 = 3 , 4
S3 = 5 - 7
S4 = 8 - 14
S5 = 15 - 29
S6 = 30+

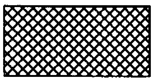
Material Definitions

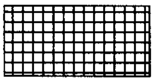

Double surface dressing

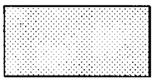

Flexible bituminous surface

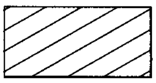

Bituminous surface
(Usually a wearing course, WC, and a basecourse, BC)


Bituminous roadbase, RB


Granular roadbase, GB1 - GB3


Granular sub-base, GS


Granular capping layer or selected subgrade fill, GC


Cement or lime-stabilised roadbase 1, CB1


Cement or lime-stabilised roadbase 2, CB2


Cement or lime-stabilised sub-base, CS

CHART 1 GRANULAR ROADBASE / SURFACE DRESSING

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

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

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

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

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

Note: Sub-base to fill substitution not permitted.

CHART 3 GRANULAR ROADBASE / SEMI-STRUCTURAL SURFACE

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

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

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

CHART 4 COMPOSITE ROADBASE / SEMI - STRUCTURAL SURFACE

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

Note: Sub-base to fill substitution not permitted.

CHART 5 GRANULAR ROADBASE / STRUCTURAL SURFACE

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

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

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

CHART 6   COMPOSITE ROADBASE / STRUCTURAL SURFACE

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

Note: Sub-base to fill substitution not permitted.

CHART 7 BITUMINOUS ROADBASE / SEMI-STRUCTURAL SURFACE

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

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

2 A cement or lime-stabilised sub-base may also be used but see Section 7.7.2.

CHART 8 CEMENTED ROADBASE /' SURFACE DRESSING

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

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

11. REFERENCES

AMERICAN SOCIETY FOR TESTING AND MATERIALS (1987) Test D559, *Annual Book of ASTM Standards,* Vol. 4.08. Philadelphia.

AMERICAN SOCIETY FOR TESTING AND MATERIALS (1990). D3910, *Annual Book of ASTM Standards, Vol.* 4.03 Philadelphia.

ANDERSON, M G and D M LLOYD (1991) Using a combined slope hydrology-stability model to develop cut slope design charts. Proc. Institution of Civil Engineers, Part 2, 91, London

ASPHALT INSTITUTE (1980). Model construction specifications for asphalt concrete and other plant-mix types. *Specification Series No I (SS-1)* The Asphalt Institute, College Park, Maryland.

ASPHALT INSTITUTE (1983). Principles of construction of hot-mix asphalt pavements *Manual Series No. 22* (MS-22). The Asphalt Institute, College Park, Maryland.

ASPHALT INSTITUTE (1988) Mix design methods for asphalt concrete and other hot-mix types *Manual Series No 2* (MS-2) The Asphalt Institute, College Park, Maryland.

ASPHALT INSTITUTE (1989). The Asphalt Handbook. *Manual Series No* 4 (MS-4) The Asphalt Institute, College Park, Maryland.

ASPHALT INSTITUTE (1991). Marshall Mix Design Criteria Changes *The Magazine of the Asphalt Institute* Vol 5, No. 3 The Asphalt Institute, College Park, Maryland.

BELL, F G (1987) Ground Engineers Reference Book. Butterworth and Co Ltd, Publishers.

BISHOP, A W (1955). The use of slip circles in the stability analysis of slopes. *GeotechnIques, Vol* 3. London.

BOFINGER, H E, H O HASSAN and R I T WILLIAMS (1978) The shrinkage of fine grained soil cement. *TRRL Supplementary Report SR* 398 Transport and Road Research Laboratory, Crowthorne.

BRUNTON, J M, S F BROWN and P S PELL (1987). Developments to the Nottingham Analytical Design , Method for Asphalt Pavements Proc 6th Int Conf. on the Structural Design of Asphalt Pavements. University of Michigan, Ann Arbor.

BULLAS, J C and G WEST (1991). Specifying clean, hard and durable aggregate for bitumen macadam roadbase *TRRL Research Report RR* 284 Transport and Road Research Laboratory, Crowthorne.

CHESHER, A and R HARRISON (1987) Vehicle operating costs: evidence from developing countries. The Highway Design and Maintenance Standards Series. The International Bank for Reconstruction and Development, Washington, DC.

CIRIA (1988) Laterite in road pavements. *Special Publication 47.* Construction Industry Research and Information Association, London, UK.

COOK, D J and P SUWANVITAYA (1982) Properties and behaviour of lime-rice husk ash cements. *UNCIV Report No* 8208. University of New South Wales, Australia.

DEPARTMENT OF TRANSPORT (1986) Specification for Highway Works. Part 2, Earthworks and Part 3, Pavements. Her Majesty's Stationery Office, London.

DICKINSON, E J (1982). The performance of thin bituminous pavement surfacings in Australia. Proc. 11th ARRB Conf 11(3) pp. 35-51.

ELLIS, C 1 (1974). Village-scale production of lime in Ghana *TRRL* Supplementary Report SR *42* Transport and Road Research Laboratory, Crowthorne.

ELLIS, C 1 (1975). Risk and pavement design decisions in developing countries. *TRRL Laboratory Report* LR 667 Transport and Road Research Laboratory, Crowthorne.

ELLIS, C I (1980). Sod compaction at low moisture contents. Field trials in the Sudan. Proc. 7th Regional Conf for Africa on Sod Mechanics and Foundation Engineering, Accra, Ghana. A A Balkema, Rotterdam.

GERRITSEN, A H and R C KOOLE (1987) Seven years experience with the structural aspects of the Shell Pavement Design Manual Proc. 6th Int Conf. on the Structural Design of Asphalt Pavements. University of Michigan, Ann Arbor.

HARRIS, V A P, L S HITCH and J M JOWETT (1983) Bituminous stabilisation of fine sands construction of the Baiomori - Gashua road, Nigeria. Proc. Institution of Civil Engineers, Part 1, 74 pp 277-300. London.

HIGHWAY RESEARCH BOARD (1962) The AASHO Road Test. Report 5, Pavement Research Highway Research Board Special Report No 61 E National Research Council, Washington, DC.

HITCH, L S (1981). Surface dressing in developing countries research in Kenya. *TRRL Laboratory Report LR* 1019, Transport and Road Research Laboratory, Crowthorne.

HITCH, L S and M STEWART (1987) The preparation of cut-back bitumens to ASTM specifications by blending readily available constituents *TRRL Research Report RR 104,* Transport and Road Research Laboratory, Crowthorne.

M HIZAM HARUN and C R JONES(1992) The performance of polymer modified asphaltic concrete on climbing lanes in Malaysia. Proc 16th Australian Road Research Board Conf. ARRB, Melbourne.

HOBAN, T (1990). Modified bitumen binders for surface dressing. Chemistry *and Industry (J.* Society of *the Chemical Industry), No* 17, pp 538-542. London.

HOWE, J D G F (1972). A review of rural traffic counting methods in developing countries. *RRL Report LR 427,* Road Research Laboratory, Crowthorne.

HOWELL, J H, J E CLARKE, C J LAWRANCE and I SUNWAR (1991). Vegetation structures for stabilising highway slopes. A manual for Nepal. Transport and Road Research Laboratory, Crowthorne.

JACKSON, G P and D BRIEN (1962) Asphaltic concrete. Shell International Petroleum Company Ltd., London.

LAWRANCE, C J, R J BYARD and P J BEAVEN (1993) The Terrain Evaluation Manual Transport Research Laboratory State of the Art Review Her Majesty's Stationery Office, London.

LEWIS, W A, R T MURRAY and I F SYMONS (1975). Settlement and stability of embankments constructed on soft alluvial sods. Proc. Institution of Civil Engineers, Part 2 London.

LIONJANGA, A V, T TOOLE and D NEWILL (1987). The development of specifications for the use of low-grade calcretes in lightly trafficked roads in Botswana *Transportation* Research *Record 1106,* Vol 1, pp 281-304. National Research Council, Washington, DC.

MEHTA, P K (1979). The chemistry of cements made from rice husk ash. Proc. of Workshop on Production of Cement-Like Materials from Agro-Wastes UNIDO-ESCAP-RCTT-PCSIR. Peshawar, Pakistan.

MILLARD, R S (1993). Roadmaking in the tropics: materials and methods. Transport Research Laboratory State of the Art Review. Her Majesty's Stationery Office, London.

NATIONAL ASSOCIATION OF AUSTRALIAN STATE ROAD AUTHORITIES (1983). Guide to the control of moisture in roads NAASRA, Sydney.

NATIONAL ASSOCIATION OF AUSTRALIAN STATE ROAD AUTHORITIES (1986). Guide to stabilisation in roadworks. NAASRA, Sydney.

NATIONAL INSTITUTE FOR TRANSPORT AND ROAD RESEARCH (1978). The Construction of road embankments. *Technical Recommendations for Highways TRH* 9. Council for Scientific and Industrial Research, Pretoria.

NATIONAL INSTITUTE FOR TRANSPORT AND ROAD RESEARCH (1984). Determination of the initial consumption of lime in sod (gravel ICL test). Test *method CA 34-83.* Council for Scientific and Industrial Research, Pretoria.

NATIONAL INSTITUTE FOR TRANSPORT AND ROAD RESEARCH (1986) Cementitious stabilisers inroad construction *Technical Recommendations for Highways TRH 13.* Council for Scientific and Industrial Research, Pretoria.

NATIONAL INSTITUTE FOR TRANSPORT AND ROAD RESEARCH (1987a). The design of road embankments. *Technical Recommendations for Highways TRH 10* Council for Scientific and Industrial Research, Pretoria.

NATIONAL INSTITUTE FOR TRANSPORT AND ROAD RESEARCH (1987b) The investigation, design, construction and maintenance of road cuttings *Technical Recommendations for Highways TRH 18.* Council for Scientific and Industrial Research, Pretoria.

NETTERBERG, F (1984). Rapid field test for carbonation of lime or cement-treated materials. National Institute for *Transport and Road* Research *Report RSl2/84.* Council for Scientific and Industrial Research, Pretoria.

NETTERBERG, F and M I PINARD (1991) Derivation of interim performance-related specifications for coarsely-graded plastic calcrete basecourse gravels Proc. 10th Regional Conference for Africa on Soil Mechanics and Foundation Engineering, Maseru.

NEWILL, D, R ROBINSON and KASSAYE AKLILU (1987) Experimental use of cinder gravel on roads in Ethiopia. Proc 9th Regional Conference for Africa on Sod Mechanics and Foundation Engineering, Lagos, Nigeria A A Balkema, Rotterdam.

O'CONNELL, M J, J H G WAMBURA and D NEWILL (1987) Soil compaction at low moisture content in dry areas in Kenya Proc. 9th Regional Conf. for Africa on Soil Mechanics and Foundation Engineering, Lagos, Nigeria. A A Balkema, Rotterdam.

PAIGE-GREEN, P (1981). Current techniques in groundwater control applied to cut slopes. *Trans Geol* Soc. S. Africa, Vol *84, pp* 161-167

PAIGE-GREEN, P, F NETTERBERG and L R SAMPSON (1990). The carbonation of chemically stabilised road construction materials A guide to its identification and treatment. Division of Roads and Transport Technology. *Research Report DPVT - 123* Council for Scientific and Industrial Research, Pretoria.

PARSLEY, L L and R ROBINSON (1982). The TRRL Road Investment Model for developing countries (RTIM 2) *TRRL Laboratory Report LR 1057,* Transport and Road Research Laboratory, Crowthorne.

PARSONS, A W (1976). The rapid measurement of moisture conditions of earthwork material *TRRL Laboratory Report LR 750.* Transport and Road Research Laboratory, Crowthorne.

PARSONS, A W (1993) Compaction of soils and granular materials a review of research performed at the Transport Research Laboratory. Her Majesty's Stationery Office, London.

PARSONS, A W and A F TOOMBS (1987) The precision of the moisture condition test *TRRL Research Report RR* 90 Transport and Road Research Laboratory, Crowthorne.

PATERSON, W D O (1987) Road deterioration and maintenance effects: models for planning and management. The Highway Design and Maintenance Standards Series. The International Bank for Reconstruction and Development, Washington, DC.

POWELL, W D, J F POTTER, H C MAYHEW and M E NUNN (1984). The structural design of bituminous roads *TRRL* Laboratory Report LR 1132. Transport and Road Research Laboratory, Crowthorne.

POWELL, W D and D LEECH (1982) Standards for compaction of dense roadbase macadam. *TRRL Supple*-mentary *Report SR* 717 Transport and Road Research Laboratory, Crowthorne.

BOLT, J, S G WILLIAMS, C R JONES and H R SMITH (1987) Performance of a full scale pavement design experiment in Jamaica. *Transportation Research Record* 1117. Transportation Research Board, Washington DC.

ROLT, J, H R SMITH and C R JONES (1986). The design and performance of bituminous overlays in tropical environments Proc 2nd Int Conf. on the Bearing Capacity of Roads and Airfields. Plymouth, UK.

RUSSAM, K and D CRONEY (1960) The moisture conditions beneath ten overseas airfields. Conf on Civil Engineering Problems Overseas, London, July 1960, Paper 9. Institution of Civil Engineers, London pp 199-206 and discussion pp 217-234.

SAM ESHIMA, T and P M BLACK (1979). Clay index - a simple method of assessing the quality and performance of roading aggregate Proc New Zealand Roading Symposium, A3, pp 1-10.

SAMPSON, L R and F NETTERBERG (1989). The durability mill. A new performance-related durability text for basecourse aggregates. The Civil *Engineer In South Africa, September 1989,* pp 287-294.

SHERWOOD, P T (1993) Soil stabilisation with lime and cement. Transport Research Laboratory State of the Art Review. Her Majesty's Stationery Office, London.

SMITH, H R, J BOLT and J H G WAMBURA (1990), The durability of bituminous overlays and wearing courses in tropical environments. Proc. 3rd Int Conf on the Bearing Capacity of Roads and Airfields. The Norwegian Institute of Technology, Trondheim, Norway.

STOCKER, P T (1972). Diffusion and diffuse cementation in lime and cement-stabilised clayey soils *Special Report No* 8 Australian Road Research Board, Victoria, Australia.

STRAUSS, P J, V P SERVAS and G P MARAIS (1984) Unexpected surface cracking of asphaltic wearing courses. Proc 4th Conf. on Asphalt Pavements for Southern Africa, Cape Town CAPSA, South Africa.

TRANSPORT AND ROAD RESEARCH LABORATORY (1976) Settlement and stability of earth embankments on soft foundations *TRRL Supplementary Report SR* 399. TRRL, Crowthorne.

TRANSPORT AND ROAD RESEARCH LABORATORY (1978) Guide to the measurement of axle loads in developing countries using a portable weighbridge *TRRL Road Note No 40* Her Majesty's Stationery Office, London.

TRANSPORT AND ROAD RESEARCH LABORATORY (1982) A guide to surface dressing in tropical and sub-tropical countries *Overseas Road Note* No 3 TRRL, Crowthorne.

TRANSPORT AND ROAD RESEARCH LABORATORY (1988). A guide to road project appraisal Overseas *Road Note No* 5, TRRL, Crowthorne.

TRANSPORT AND ROAD RESEARCH LABORATORY (1988) A guide to geometric design Overseas *Road* Note *No 6.* TRRL, Crowthorne

US ARMY CORPS OF ENGINEERS (1991) Hot-mix asphalt paving handbook US Army Corps of Engineers and National Asphalt Pavement Association. Vicksberg and Maryland, USA.

WATANATADA, T. C G HARRAL, W D O PATERSON, A M DHARESHWAR, A BHANDARI and K TSUNOKAWA (1987) *The Highway Design and Maintenance Standards Model Volume 1, Description of the HDM-111 Model Volume 2,* User's manual for the HDM-III Model Highway Design and Maintenance Standards Series, The International Bank for Reconstruction and Development, Washington, DC.

WEINERT, H H (1980) The natural road construction materials of Southern Africa Academica, Pretoria.

WHITEOAK, D (1990) The Shell Bitumen Handbook. Shell Bitumen UK, London.

WILLIAMS, R I T (1986) Cement treated pavements Elsevier Applied Science Publishers, London.

12. BIBLIOGRAPHY

CHEN, F H (1988) Foundations on expansive soils. Pub Elsevier.

CRONEY, D and P CRONEY (1991). The design and performance of road pavements. Second Edition. McGraw Hill International. Maidenhead, UK.

DICKINSON, E J (1985). Bituminous roads in Australia. Australian Road Research Board, Victoria.

HEAD, K H (1984). Manual of soil laboratory testing. ELE International Ltd. Pub. Pentech Press, London and Plymouth

INGLES, O G AND J B METCALF (1972) Soil stabilization - Principles and practice. Butterworths, Sydney, Melbourne and Brisbane.

LIME STABILISATION (1987) State of the Art Report No 5. Transportation Research Board, Washington, "DC.

LIME STABILISATION MANUAL (1986). British Aggregate and Construction Materials Industries, London.

NATIONAL ASSOCIATION OF AUSTRALIAN STATE ROAD AUTHORITIES (1987). Pavement design. A guide to the structural design of road pavements. NAASRA, Sydney.

SHELL INTERNATIONAL PETROLEUM CO. (1978). Shell Pavement Design Manual London.

APPENDIX A: APPLICABLE BRITISH STANDARDS

The British Standards Institution is the independent national body for the preparation of British Standards. Enquiries should be addressed to the BSI, Linford Wood, Milton Keynes, MK14 6LE.

BS 63	Part 2 (1987)	Single-sized aggregate for surface dressing.
BS434		Bitumen road emulsions (anionic and cationic).
	Part 1 (1984)	Bitumen road emulsions.
	Part 2 (1984)	Code of practice for use of bitumen road emulsions.
BS594		Hot rolled asphalt for roads and other paved areas.
	Part 1 (1985)	Constituent materials and asphalt mixtures.
	Part 2 (1985)	Transport, laying and compaction of rolled asphalt.
BS598		Sampling and examination of bituminous mixtures for roads and other paved areas.
	Part 2 (1974)	Methods for analytical testing.
	Part 3 (1985)	Methods for design and physical testing.
	Part 100 (1987)	Methods for sampling for analysis.
	Part 101 (1987)	Methods for preparatory treatment of samples for analysis.
	Part 102 (1989)	Analytical test methods.
	Part 104 (1989)	Methods of test for the determination of density and compaction.
	Part 107 (1990)	Methods of test for the determination of the composition of design wearing course rolled asphalt.
	Part 109 (1990)	Methods for the assessment of the compaction performance of a roller and recommended procedures for the measurement of the temperature of bituminous mixtures.
BS812		Testing aggregates.
	Part 1 (1975)	Methods of determining particle size and shape.
	Part 2 (1975)	Physical properties.
	Part 100 (1990)	General requirements for apparatus and calibration.
	Part 101 (1984)	Guide to sampling and test procedures.
	Part 102 (1989)	Methods of sampling.
	Part 103 (1985)	Methods for determination of particle size distribution.
	Section 103.2(1989)	Sedimentation test.
	Part 105	Methods for determination of particle shape.
	Section 105.1 (1989)	Flakiness index.
	Part 109 (1990)	Methods of determination of moisture content.
	Part 110 (1990)	Method for determination of aggregate crushing value (ACV).
	Part 111 (1990)	Methods for determination of ten per cent fines value (TFV).

	Part 112 (1990)	Methods for determination of aggregate impact value (AIV).
	Part 113 (1990)	Method for determination of aggregate abrasion value.
	Part 114 (1989)	Method for determination of the polished-stone value.
	Part 117 (1988)	Method for determination of water soluble chloride salts.
	Part 118 (1988)	Methods for determination of sulphate content.
	Part 121 (1989)	Method for determination of soundness.
BS890 (1972)		Building limes.
BS1377 (1990)		Methods of test for soils for civil engineering purposes.
	Part 1	General requirements and sample preparation.
	Part 2	Classification tests.
	Part 3	Chemical and electro-chemical tests.
	Part 4	Compaction related tests.
	Part 5	Compressibility, permeability and durability tests.
	Part 6	Consolidation and permeability tests in hydraulic cells and with pore pressure measurement.
	Part 7	Shear strength tests (total stress).
	Part 8	Shear strength tests (effective stress).
	Part 9	In situ tests.
BS1924 (1990)		Stabilised materials for civil engineering purposes.
	Part 1	General requirements, sampling, sample preparation and tests on materials before stabilisation.
	Part 2	Methods of test for cement-stabilised and lime-stabilised materials.
BS2000		Petroleum and its products.
	Part 49 (1983)	Penetration of bituminous materials.
	Part 58 (1988)	Softening point of bitumen (ring and ball).
	Part 72 (1988)	Viscosity of cutback bitumen.
	Part 105 (1991)	Recovery of bituminous binders by dichloromethane extraction.
BS3690		Bitumens for building and civil engineering.
	Part 1 (1989)	Bitumens for roads and other paved areas.
BS4987		Coated macadam for roads and other paved areas.
	Part 1 (1988)	Specifications for constituent materials and for mixtures.
	Part 2 (1988)	Transport, laying and compaction.

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 B1.

TABLE B1

Correction factors for soil type (PI) used in calculating the effective depth of the water table

PI	Correction factor SF
0	0
10	0.3
15	0.55
20	0.80
25	1.1
30	1.4
35	1.6
>35	2.0

To calculate the effective depth **D** which is used to determine the applied suction in the pressure plate extractor, the following equation is used:

$$\mathbf{D = WT + (SF \times t)}$$

Where WT = depth of water table below subgrade (at its highest expected seasonal level),

SF = correction factor from Table B1,

t = pavement thickness,
with consistent units for WT, t, 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 C1, 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 C2). 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 C3.

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 roadbases 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 of weakly stabilised materials. It is more difficult to penetrate strongly stabilised 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 (TRRL (1990)) that is designed to assist with the interpretation and presentation of DCP data.

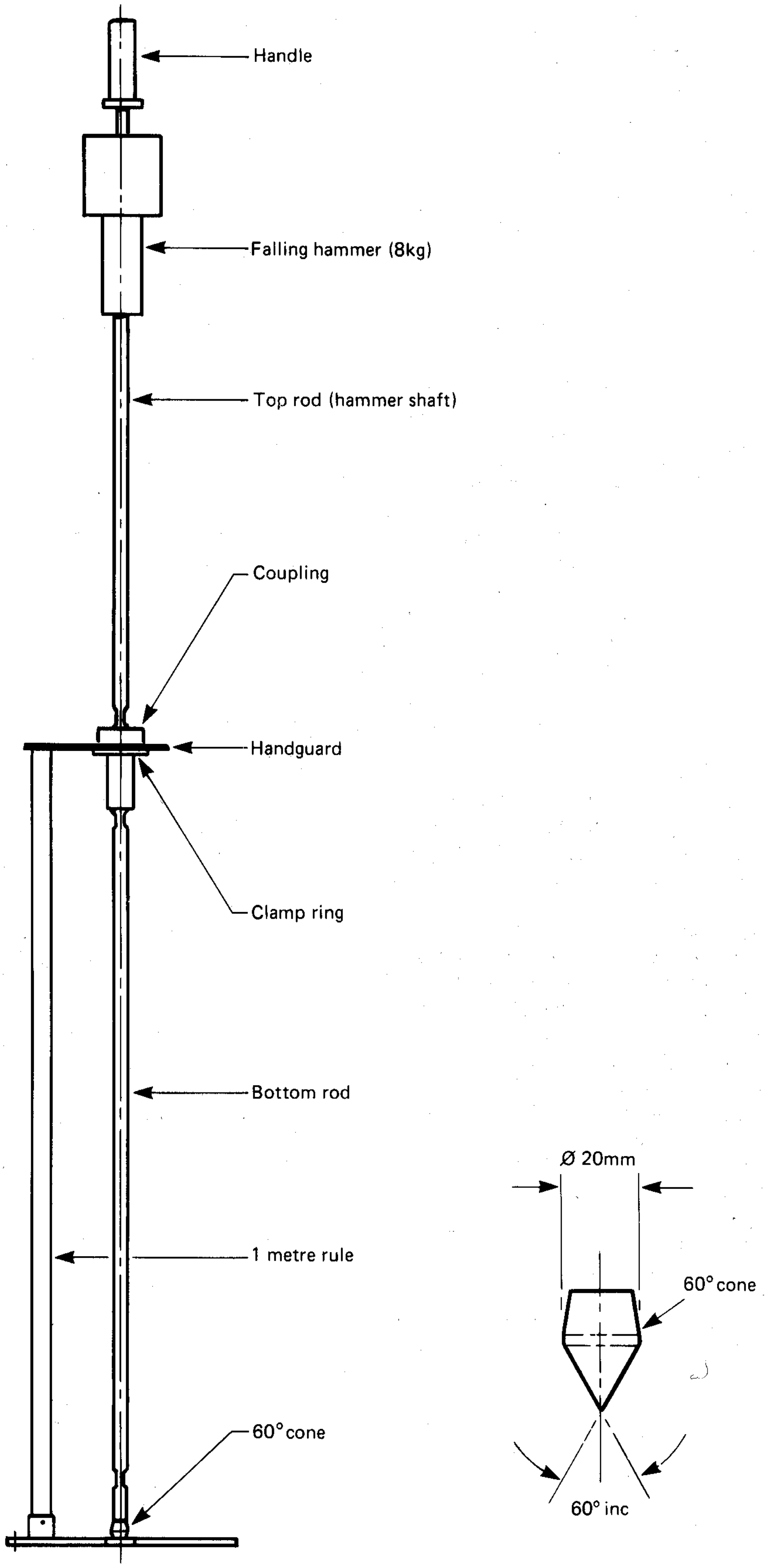


Fig.C1 TRL Dynamic cone penetrometer

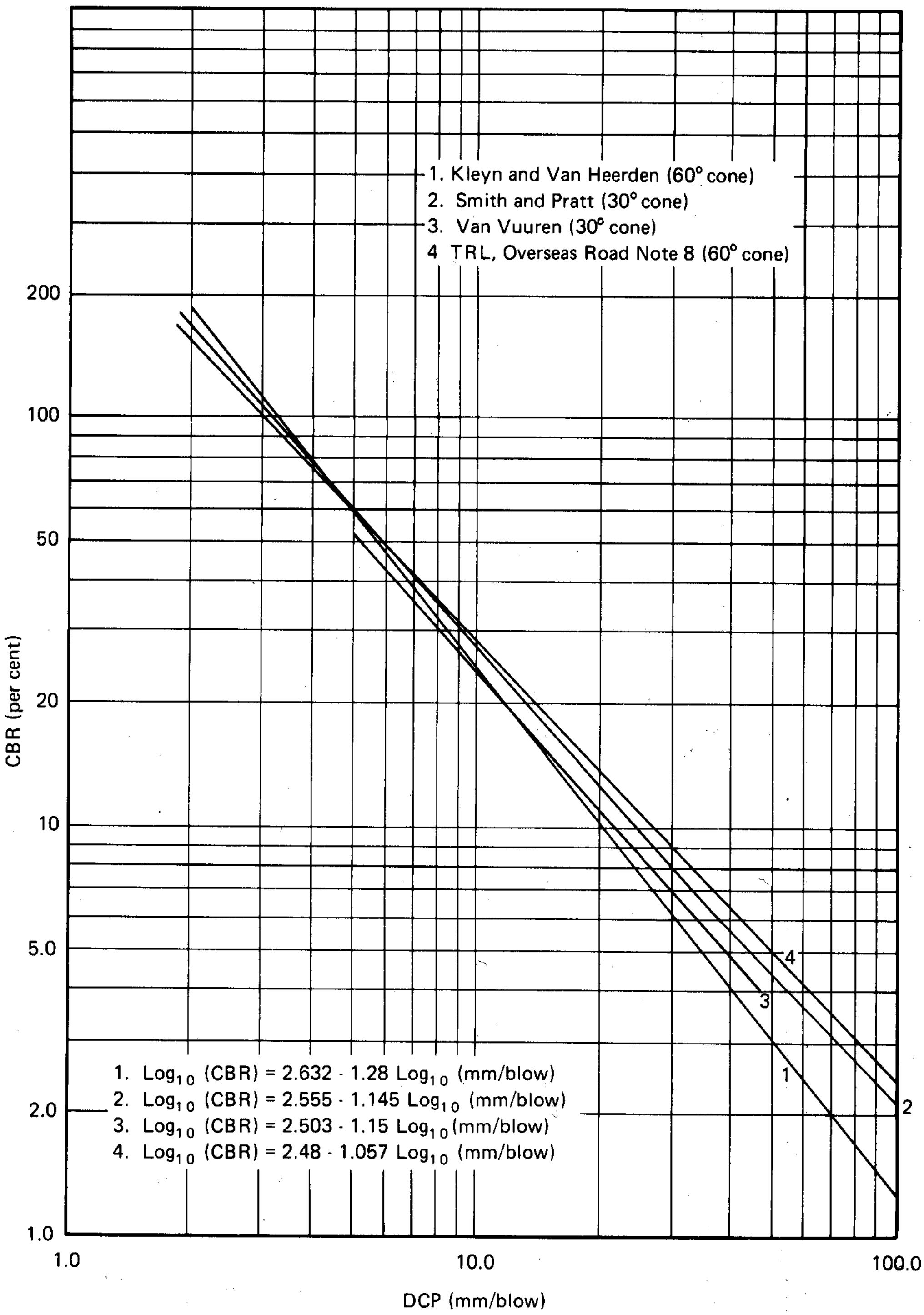


Fig.C2 DCP-CBR relationships

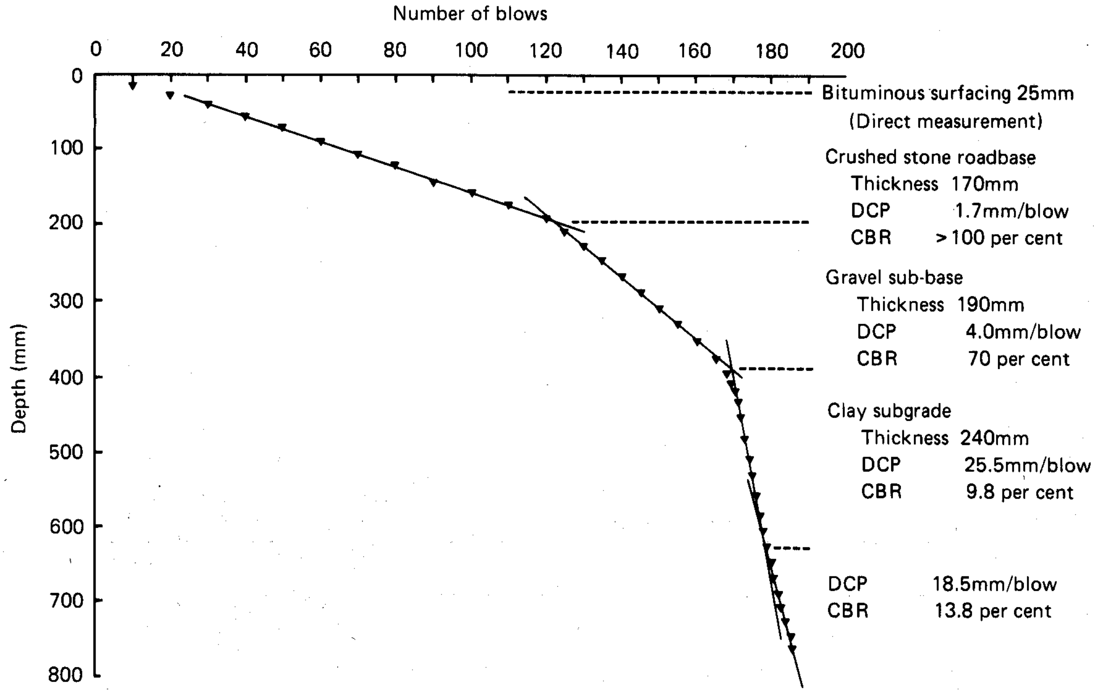


Fig.C3 DCP test result

APPENDIX D: REFUSAL DENSITY DESIGN

1 INTRODUCTION

Some authorities have adopted a procedure of extended Marshall compaction 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.

2 EXTENDED MARSHALL COMPACTION

For severe sites, the basecourse gradings, BC1 and BC2, given in Table 8.3 and Table 8.6 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 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 contents 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

3 EXTENDED VIBRATING HAMMER COMPACTION

3.1 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 ± YC 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,E,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.

3.2 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 regime 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 (3 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

4 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 PROBE PENETRATION TEST

1 GENERAL DESCRIPTION

This test utilises a modified sod 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 4 mm diameter probe rod with a hemispherical tip made of hardened steel. The probe is forced into the road surface under a load of 35 kgf (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 their equivalents at a standard temperature of WC.

2 METHOD OF OPERATION

All measurements are made in the nearside wheelpath of each traffic lane where maximum embedment of chippings can be expected. A minimum of ten measurements are required at each location. These should be evenly spaced along the road at intervals of 0 5m with 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 centred 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 centred on the measurement mark and a pressure of 35 kgf 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 millimetres. Sometimes 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. In many tropical countries this will limit probe testing to the early morning. The probe readings are corrected to a standard temperature of 30°C using Figure E1, 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 penetration values are shown in Chapter 9 2.

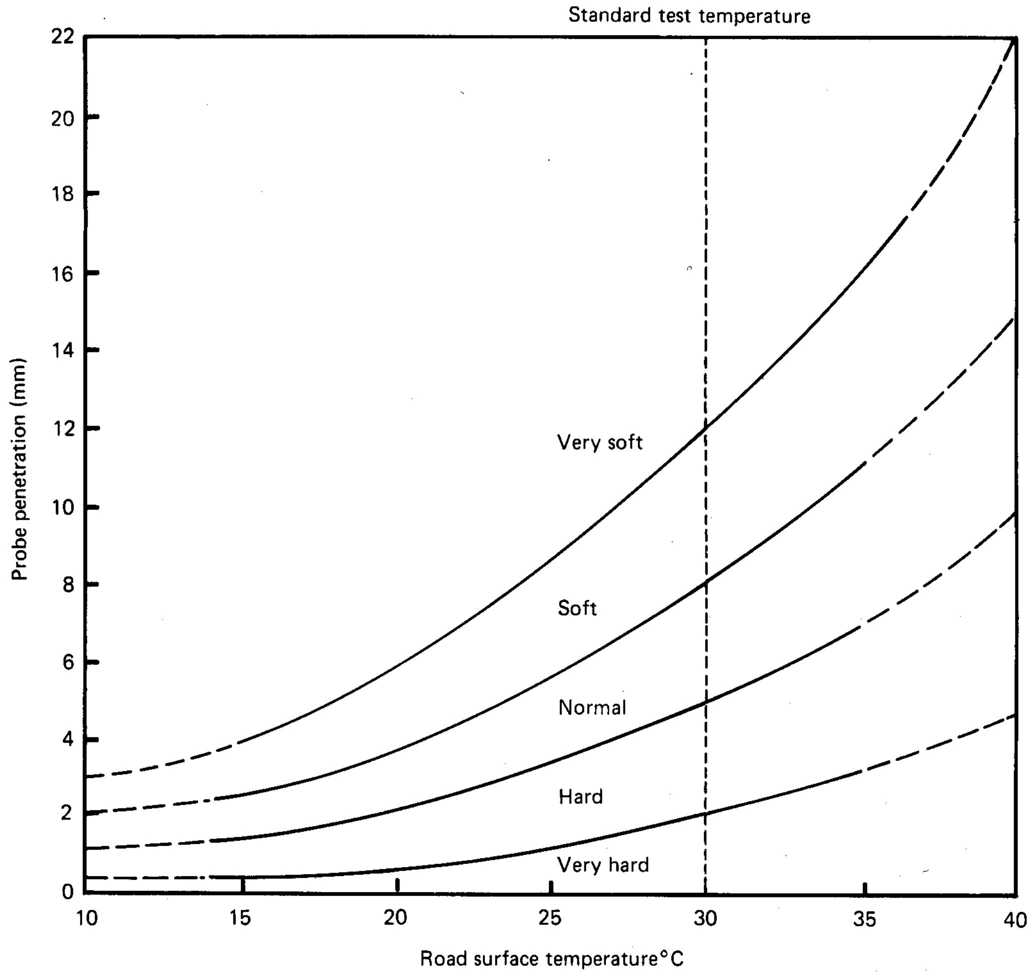


Fig.E1 Correction of road surface hardness to the standard test temperature of 30° C

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A guide to the structural design of bitumen-surfaced roads in tropical and sub-tropical countries

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