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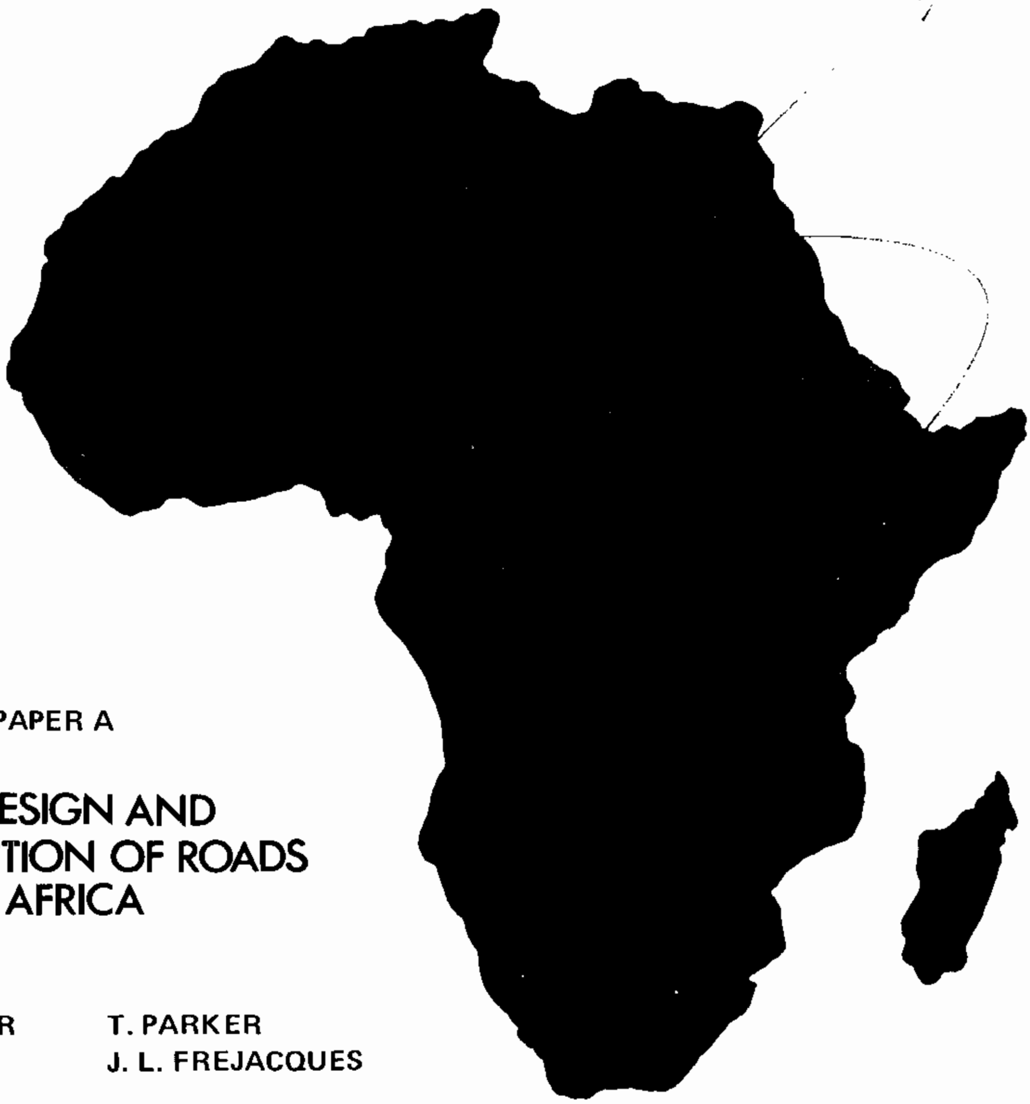

PAPER A

THE DESIGN AND CONSTRUCTION OF ROADS IN AFRICA

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THE DESIGN AND CONSTRUCTION OF ROADS IN AFRICA

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PAPER A - THE DESIGN AND CONSTRUCTION OF ROADS IN AFRICA

ABSTRACT

This paper summarises methods that have been used to design and construct road pavements in Africa. It draws heavily on the experience of British and French engineers who have observed the behaviour of large lengths of roads in many African countries.

Earth roads are treated briefly, only because there has been little scientific investigation of their behaviour. The selection of materials for gravel-surfaced roads is discussed in more detail and typical specifications of suitable materials have been included.

A large proportion of the paper has been devoted to bitumen-surface roads because the capital cost of these roads is high compared with the other two types and failures can be expensive to rectify. The concept of stage construction is described and two methods of designing the thicknesses of pavement layers have been included.

The first design system is a flexible method based on the cumulative number of equivalent standard axles which the pavement is to carry. This system can be used in all developing countries because the design method considers the damage caused by the magnitude of the individual axle-load in the spectrum of axle-loads that the pavement must carry. A simplified design table, which takes account of the number of commercial vehicles or the total number of vehicles using the road, has also been included. Each design method assesses the subgrade strength when its moisture content is equal to the maximum that is likely to occur during the life of the pavement.

The traditional materials that have had wide use in the bases of sealed pavements are described, together with methods of stabilizing sub-standard materials.

The paper concludes with a short discussion on the drainage of pavement layers.

Introduction

To develop an effective transport system throughout Africa there is a need for roads of all standards from engineered earth and gravel surfaced roads to those with high quality bitumen surfaces. This paper considers the methods that have been used for designing and constructing these roads in countries with tropical and sub-tropical climates.

The major proportion of research into the design of pavements has been carried out in Europe and America where the climatic, traffic and economic conditions are different from those in Africa. It is not surprising that the methods of design commonly used in Europe and America are not always applicable in African countries and can lead to unnecessarily expensive road pavements.

There are three main differences between the design of road pavements for developing countries in hot climates and developed countries in temperate climates

1. The spectrum and volume of axle loads to which the pavement will be subjected can be completely different, ie the basic load to be carried by the road is different.

2. Subgrades in hot climates are generally drier and stronger than those in temperate climates and need a smaller total pavement thickness to protect them.

3. The growth of traffic in developing countries is difficult to predict. Therefore, the estimation of traffic volumes, even in the near future can be inaccurate, causing a similar error in the design life of the pavement.

Methods by which account can be taken of these three differences are discussed in the paper. By quantifying the damage to the road structure attributable to axle loads of various magnitudes and relating this damage to "equivalent standard axles", a single chart can be used to design sealed pavements, irrespective of the magnitude of the maximum axle-load limit permitted on the pavement. Pavements are designed to accept a certain amount of damage during their design lives, before strengthening becomes necessary.

It is common practice in temperate climates to measure the strength of the subgrade after 4 days of soaking because there is a high probability that it will be nearly saturated for a significant period during its service life. In arid and semi-arid areas, the probability that the subgrade will become saturated is very low. Experience has shown that, in all but the flat, low-lying areas of wet tropical regions, regular attention to the maintenance of suitable drainage will ensure that the subgrade rarely becomes wetter than a maximum which has been found to correspond to the optimum moisture content for compaction (BS light, or Std AASHO). There is an increasing tendency in hot climates to determine the maximum moisture content that is likely to develop in the subgrade and to design the thickness of the pavement on the basis of the strength of the subgrade at this moisture content.

In many African countries traffic volumes have increased enormously over the last 20 years and it is difficult to predict with any accuracy how traffic will develop on individual roads. For this reason and to make good immediate use of resources of available capital, stage construction is frequently employed. With this system it is not necessary to make the large capital outlays which are required if the road is to withstand the cumulative traffic volume expected during a design life

of 20 years, particularly when the growth rate is high. A gravel-surfaced road constructed from plastic gravels can give adequate service for a number of years before the volume of traffic warrants the addition of a base and surface dressing. Further strengthening can be carried out when required by the addition of structural overlays.

The use of stabilized base materials is important when bitumen-surfaced roads are required in areas where suitable natural materials for base construction are not available within economical haulage distance. Lime, cement and bitumen stabilization are discussed and some notes on the small-scale production (ie village industry production) of lime are included.

Earth Roads

Little has been published on the design and building of earth roads because the improvements in the techniques have been the result of field experimentation. References 1 and 2 cover the most important points. In practice, each type of soil must be used so that the earth road remains open to traffic for the longest period in each year.

Soils with some cohesion can provide an adequate road for vehicles in dry weather, but , when they are wet, they lose most of their strength and are unable to support the weight of vehicles. To keep these roads open for the maximum amount of time, it is essential that their drainage is properly engineered. The most important factor in the construction of earth tracks and roads is to provide a cross-section that will be least affected by rain. Where it is possible, low-lying or swampy ground must be avoided because low embankments are necessary even for simple tracks.

Earth roads warrant only a minimum of earthworks which normally are carried out by hand or simple dozing operations. The importation of pavement materials is not justified economically and the materials are taken from the line of the road or immediately adjacent to it. When the material is suitable, earth cut to form the side drains should be used to build up the formation above the surrounding ground level. Maintenance operations must protect this formation, ie material that has moved to the sides under the action of traffic is bladed back to the centre of the road to maintain the high formation.

The geometrical standards for these roads are governed by the mechanical capabilities of the vehicles using them and the volume of traffic. Standards have been suggested by Mellier(2) (page 30) but a further discussion of the selection of road standards for Africa is given in Theme F and the subject is not considered in this paper.

Gravel-surfaced Roads

The next improvement to the strength of a road pavement can be made by adding a layer of gravel which will retain a considerable proportion of its strength when it is wet. Before this work is carried out, the geometry of the road should be checked to ensure that it is of adequate standard for the volume of traffic that will be using the road(2).

The type of gravel that is considered to be suitable for these roads varies from one country to another, but the individual specifications reflect the requirements of the particular climate.

There are two components which make up the shear strength of natural materials, namely, the internal friction and the cohesion (Fig 1). The frictional strength is obtained from the interlocking of the granular particles but its magnitude is proportional to the confining pressure holding the particles together. Water has very little effect on the frictional component provided that the pore pressure is low.

The cohesion of the material is the primary strength of the fine fraction and is independent of the confining pressure. However, it can change markedly with small changes in the amount of water present. When the material is saturated, the cohesion is a minimum and this is normally referred to as the "true" cohesion. As the water content is reduced, the capillary forces increase and the "apparent" cohesion rises.

One must compromise when the gravel for a road surface is selected. The material should be strong when it is wet, allowing the road to be open in all weather, and strong when it is dry, giving an adequate resistance to traffic wear with a low cost of maintenance. Wet strength is obtained by good aggregate interlock which increases with the angularity of the granular fraction. The interlock of gravels with rounded aggregate can be improved by crushing. Some soil binder is essential to mobilise the strength of the aggregate interlock at the surface but, in wet climates, it is kept to a minimum and the road surface is kept as dry as possible.

In dry climates, a greater percentage of soil binder with an increased Plasticity Index (PI) is needed to ensure that the surface is sufficiently strong.

An ideal soil binder for gravel pavements will have good cohesion, will resist penetration by water thus retaining its cohesion during storms and will not swell or shrink excessively with wetting and drying.

Limits of grading of gravel for surfacings have been suggested(3), and these are reproduced in Table 1.

TABLE 1

Limits of particle-size distribution for bases and surfacings

BS sieve size	Percentage passing					
	Base			Surfacing	Base or surfacing	
	Nominal maximum size			Nominal maximum size	Nominal maximum size	
	75mm (3in)	38 mm($1\frac{1}{2}$in)	19mm($\frac{3}{4}$ in)	19mm($\frac{3}{4}$ in)	9.5mm($\frac{3}{8}$ in)	4.75mm (3/16in)
75 mm	100	-	-	-	-	-
38 mm	80 - 100	100	-	-	-	-
19 mm	60 - 80	80 - 100	100	100	-	-
9.5 mm	45 - 65	55 - 80	80 - 100	80 - 100	100	-
4.75 mm	30 - 50	40 - 60	50 - 75	60 - 85	80 - 100	100
2.36 mm	-	30 - 50	35 - 60	45 - 70	50 - 80	80 - 100
1.18 mm	-	-	-	35 - 60	40 - 65	50 - 80
600 μm	10 - 30	15 - 30	15 - 35	-	-	30 - 60
300 μm	-	-	-	20 - 40	20 - 40	20 - 45
75 μm	5 - 15	5 - 15	5 - 15	10 - 25	10 - 25	10 - 25

Notes

1. Not less than 10 per cent should be retained between each pair of successive sieves specified for use, excepting the largest pair.

2. The two smaller sized materials (9.5 and 4.75 mm) may have up to 35 per cent of stones not larger than 38 mm, provided that the material passing the 4.75 mm sieve is within the limits specified.

3. The material passing the 425 μm sieve shall have the plasticity characteristics given in Table 2 when used for surfacing gravel roads.

Because of the wide range of climatic conditions throughout Africa there is a wide range of plasticities that are accepted in gravel surfacings. It must be remembered that only the minus 0.4 mm fraction is used in the PI determination and the percentage of this fraction in the total material is important. Low Cost Roads(1) suggests the following general limits.

TABLE 2

Plasticity characteristics preferred for gravel surfacings

Climate	Liquid limit not to exceed (%)	Plasticity index range (%)	Linear shrinkage (%)
Moist temperate and wet tropical	35	4-9	2-4
Seasonal wet tropical	40	6-15	3-7
Arid	55	15-30	7-14

On the other hand Mellier(2) suggests that, in wet tropical climates the plasticity index should be between 8 and 12, in seasonally wet tropical climates the index should be between 10 and 20, and in arid climates it should be between 15 and 30.

Particular specifications for road gravel materials used by some African countries are included in Table 3 below.

TABLE 3

	Senegal	Cote d'Ivoire	Cameroun	Congo Gabon	RCA	Northern Nigeria
Lateritic gravel						
Percentage passing 2 mm	<60	30-65	20-70	<50	40-70	
Percentage passing 80 μm	20-35	16-30	20-35	<40	20-40	
PI	10-25	15-28	15-30	<40	10-25	
Group		A2-6/A2-7		A2-6/A7-5	A2-6/A2-7	
Group Index		0-2		0-2	0-2	

TABLE 3 (Continued)

	Senegal	Cote d'Ivoire	Cameroun	Congo Gabon	RCA	Northern Nigeria
<u>Sands</u>						
Percentage passing 2 mm			20-60			
Percentage passing 80 µm		26-40	15-30			
PI		5-20	10-25			
Group		A2-6/A2-7	A2-6/A2-7			
Group Index		0-2	0-2			
<u>Clay sands</u>						
Liquid limit		16-32		20-35		40
PI		5-15		3-15		(10-15 desirable (5-12 necessary
Group Index		0-1		0-1		
Soaked CBR at 95% Modified Proctor	60	60	40	40	60	

Other authorities (4) state that in Cameroun the Plasticity Index is kept between 8 and 18. In the Sahara, different regions have different specifications. Where the rainfall is 50 to 100mm/a gravel with a PI approaching 12 is used. In the dry South zone, there is no limit on the plasticity and, if the PI is less than 4, fines are added or the material is stabilized.

In Niger(4) a PI between 8 and 20 is generally acceptable while the CBR after 24h soaking is of the order of 50 to 60 per cent.

Elsewhere in the world, other specifications have been developed to provide a material that is suitable for that particular climate(5) (6).

It is not common to find natural gravels that comply with the specification in all respects. If it is necessary to haul gravel for long distances, it is often more economical to improve the quality of a local, sub-standard gravel by crushing, screening or blending with other local materials.

Gravels which contain an excessive proportion of large material or oversized aggregate may be crushed to improve both the grading and the interlock (or frictional strength). Sometimes crushing can be carried out by rolling in situ, but this will depend on the type of roller used and the toughness of the aggregate. Heavy grid rollers can be used successfully to break up the oversized aggregate in many natural gravels, soft rocks and decomposing rocks.

Materials that are dry can have their grading improved by screening out excessively coarse or fine materials from the gravel. Of course, this is not possible if the material is damp.

Gravels that are deficient in certain fractions may have other materials blended with them so that the combined gravel has acceptable properties. For example, gravels which have insufficient binder can be improved by adding a clayey material that is rich in binder. Expensive mixing equipment is not usually required and the operation can be completed economically by a grader.

It is possible to improve sub-standard materials by adding stabilizing agents such as cement, lime or bitumen but the expense of this operation is not warranted for roads with gravel surfaces.

Thickness Design

The estimation of the thickness of pavement layers in gravel roads is much less critical than in bitumen-surfaced roads, because one is usually dealing with lower volumes of traffic and deteriorated sections are relatively inexpensive to repair.

In many instances, no individual design is carried out and engineers use a standard thickness of pavement based on general experience in the area. Mellier(2) has included a simple design chart that was developed a number of years ago for sealed pavements (Fig 2). It can be used as a guide for estimating the thickness of gravel-surfaced pavements in conditions about which the engineer is doubtful.

Bitumen-surfaced roads

When a sealing layer is added to the surface of a gravel road, the capillaries through which water reaches the surface and evaporates are closed and the moisture content in the top of the base may rise. Obviously the change in the moisture content will depend on the climate, the geometry and the existence and position of a water-table. In the wetter regions, where a plastic gravel may have been satisfactory for a number of years as an unsurfaced pavement, the provision of a seal will reduce the "apparent cohesion" of the soil binder, weaken the base and possibly cause its failure.

If a gravel road is to be upgraded to a bitumen-surfaced road, the materials in the existing pavement must be closely examined to ensure that they will be satisfactory in the layers that they will form in the new road, even though the gravel road may have provided excellent service for a long period. The selection of the materials and the design of the thickness must be carried out more carefully.

The traffic to be carried by a road can be classified in terms of the equivalent number of repetitions of a "standard axle" load or in terms of the number of commercial vehicles. The first system is used by the TRRL in LR 279 and in a draft revision of RN 31(7) while the CEBTP use the latter system for the design of roads in tropical countries(8).

Private cars cause insignificant damage to pavements when compared with commercial vehicles and it is normal to consider only the total number and the axle-loading of the commercial vehicles that will use the road during its design life.

It can be very difficult to predict the rate at which traffic will increase in developing countries and, consequently, the cumulative traffic loading for which the road must be designed. If regular traffic censuses have been carried out, some indication of the growth rate can be obtained but extrapolation over long design periods of 20 years can cause large errors in the predictions. When there is little or no traffic data available, less accurate predictions of traffic growth can be made by extrapolating the rate at which vehicle numbers are increasing in the country and the rise in the consumption of fuel.

Because of the uncertainty of traffic forecasts in developing countries, it is more efficient to design the pavement for a short design life of 10 years or less. Strengthening layers can be added when the residual strength of the pavement is approaching insufficiency to carry the traffic that is using the road.

The damage caused by the passage of vehicles over a road is related to the magnitude of the axle loads and the number of times they are applied. In the design methods which are based on the "equivalent standard axle" concept, the damage caused by each of the axle loads in a typical spectrum is expressed in terms of the damage that would be caused by a "standard axle". A standard axle of 8200 kg (18 000 lb) is used in British practice.

Equivalence factors derived from some of the results of the AASHO road test can be used with care for the majority of roads in developing countries(7). These convert the number of axles in different load categories to the equivalent number of 8200 kg (18 000 lb) axles and they are reproduced in Table 4.

The axle-load spectrum for commercial vehicles in a given country does not usually vary significantly from year to year unless there is a change in the type of vehicles using the roads or in the type of freight they carry. It is normal to assume that the axle-load spectrum remains constant during the design life of the road, unless these special circumstances are likely to occur.

Axle-load data can be obtained from surveys of commercial vehicles on existing roads of the same type. Various wheel-weighing devices have been used in these types of surveys but a portable machine(9), which can be used by a three-man team to weigh up to 500 vehicles per day on the roadside, has been found to be very useful. Surveys carried out in a number of countries have shown that the damaging effect of a typical sample of commercial vehicles can range from less than half a standard axle per vehicle to greater than 18 standard axles per vehicle. It follows that extrapolation between countries can introduce unacceptable errors.

When the cumulative number of equivalent standard axles in each direction has been calculated for the design life of the road, the pavement can be designed to provide the necessary carrying capacity over the subgrade by using the chart in Fig 3(7).

This design chart is divided at a point where the cumulative number of equivalent standard axles is 0.5 million. If the cumulative traffic that the road is expected to carry during its design life is less than this, a base 150 mm thick is provided and it is protected by a double surface dressing. The overall thickness of the pavement is varied for different traffic requirements by changing the thickness of the sub-base. When the cumulative number of equivalent standard axles is greater than 0.5 million, two alternatives are suggested. The first is a 150 mm base protected by 50 mm of premix bituminous surfacing and the second is a base 200 mm thick covered by a double surface dressing.

A simpler method, which assumes that the damage caused by traffic is directly related to the number of commercial vehicles using the road, is preferred by the CEBTP(8). The traffic is grouped into four categories namely:

T_1 = 100 to 300 vehicles per day

T_2 = 300 to 1000 vehicles per day

T_3 = 1000 to 3000 vehicles per day

T_4 = 3000 to 6000 vehicles per day

It is assumed that the heavy vehicles comprise approximately 30 per cent of the total volume of traffic. Standard pavement designs have been produced for each category of traffic and for each of five classified groups of subgrade strengths.

TABLE 4

Factors for converting numbers of single axles and tandem-axle sets to the equivalent number of standard 8200 kg (18000 lb) axles

Axle load		Equivalence factor	
kg	lb	Single axles	Tandem-axle sets
910	2 000	0.0002	-
1 810	4 000	0.0025	-
2 720	6 000	0.01	0.0009
3 630	8 000	0.04	0.0027
4 540	10 000	0.08	0.01
5 440	12 000	0.2	0.02
6 350	14 000	0.3	0.03
7 260	16 000	0.6	0.05
8 160	18 000	1.0	0.08
9 070	20 000	1.6	0.11
9 980	22 000	2.4	0.17
10 890	24 000	3.6	0.24
11 790	26 000	5.2	0.34
12 700	28 000	7.2	0.47
13 610	30 000	9.9	0.63
14 520	32 000	13.3	0.82
15 430	34 000	17.6	1.1
16 320	36 000	22.9	1.4
17 230	38 000	29.4	1.8
18 140	40 000	37.3	2.2
19 070	42 000	47	2.7
19 980	44 000	58	3.3
20 880	46 000	72	4.1
21 790	48 000	87	4.9

Notes:

1. The factors given are those derived by Liddle(10) for this type of flexible pavement.

2. A tandem-axle set is defined as a pair of axles whose centres are less than 1.5 m (4ft 8in) apart.

3. The load on a tandem-axle set is defined as the gross load on the two axles added together.

The CEBTP design table for determining the thickness of the pavement layers is reproduced in Table 5.

TABLE 5

CEBTP table for the design of pavements

Traffic / CBR of subgrade	T_1 (100-300 v/day)		T_2 (300-1000 v/day)		T_3 (1000-3000 v/day)		T_4 (3000-6000 v/day)	
	Sub-base cm	Base cm	Sub-base cm	Base cm	Sub-base cm	Base cm	Sub-base cm	Base cm
5-10	20	15	25	15	25	20	30	20
10-15	15	15	20	15	20	20	25	20
15-30	10	15	15	15	15	20	20	20
30-80	0	15	0	15	0	20	0	20
>80	0	0	0	0	0	0	0	0
Type of surface	Type I (2 cm)		Type II (3 cm)		Type III (4 cm)		Type IV (5 cm)	

	Recommended formula	Alternatives
Type I	single sanded layer then single maintenance layer	3 cm sand asphalt or 2.5 cm dense graded mix
Type II	double sanded layer then single maintenance layer	3.5 cm sand asphalt or 3 cm dense graded mix
Type III	double sanded layer then 2.5 cm dense graded mix	4 cm dense graded mix
Type IV	double sanded layer then 3 cm maintenance layer	5 cm dense graded mix

Note:

The CBR of the subgrade is measured at the moisture content that is appropriate for the particular climatic and topographical conditions.

This table has been developed on the assumption that the legal axle-load limit is 13 tonnes and that up to 10 per cent of the commercial vehicles may exceed this figure.

The advantage of using the simpler design method must be considered in relation to the possible errors that may be introduced and it should be remembered that it is only valid in countries where the damage caused by a given number of commercial vehicles is similar to that in countries for which the method was developed. Generally, it should only be applied in areas where a 13 tonne axle limit exists.

An advantage of the equivalent standard axle-load method is its versatility because the design is not related to a given maximum axle load but can be used for any spectrum of axle loads provided that an axle-load survey has been conducted. Obviously this entails some further work but, in circumstances where there is no reliable data on which to base the design, the additional investigation is essential if one is to provide an economical structure.

The subgrade

Many methods have been used to determine the strength of the subgrade but the CBR method has been found to be reliable for a wide variety of climates and soil types.

The strength of the subgrade materials usually varies with the moisture content and the degree of compaction. It is essential that the CBR of the subgrade is determined at a moisture content and density appropriate for the particular road project. In Europe and North America it is common practice to measure the subgrade strength on specimens which have been soaked for 4 days but this would lead to designs which are too thick for the majority of roads in tropical climates. Subgrades in the tropics rarely become saturated except when the road crosses low-lying ground or the drainage is poor. Only in these bad conditions should the subgrade material be tested in the soaked state.

The subgrade should be tested at the highest moisture content that is likely to occur after the road is opened to traffic. It is convenient to classify the subgrade moisture condition in relation to the effect that the water-table has on it and to the effect of the prevailing climate.

I. Subgrades where the water-table is sufficiently close to the surface to control the moisture content.

II. Subgrades not affected by the water-table in areas with a rainfall greater than 250 mm (10 in) per annum.

III. Subgrades in arid areas with a rainfall less than 250 mm (10 in) per annum.

The depth at which the water-table is likely to affect the moisture content of the subgrade depends on the type of soil. With a non-plastic soil, the water-table would normally have to be within 1 m of the surface to affect the condition of the subgrade. However, in a sandy clay (PI approximately 20 per cent) or a heavy clay (PI approximately 40 per cent) the water-table can control the moisture content if it is within 3 m or 7 m respectively.

When the water-table is the controlling factor, the maximum moisture content likely to occur can be estimated from a knowledge of the highest level to which the water-table will rise and the relationship between the soil-moisture suction and the moisture content of the subgrade soil.

If equipment is not available to measure the soil-moisture suction, a good estimate of the highest moisture content can be obtained by measuring the moisture content of the subgrade under the centre of an existing two-lane bituminous-surfaced road with similar water-table and climatic conditions. The road should have been completed for at least two years. For these conditions, the ratio of the subgrade moisture content to the plastic limit remains sensibly constant so that allowance can be made for different soil types.

In instances where it is not possible to make a comparison with constructed pavements, an estimate of the minimum CBR of the subgrade can be obtained from Table 6.

TABLE 6

Estimated minimum design subgrade CBR values under paved roads for subgrades compacted to 95 per cent of British Standard maximum dry density

Depth of water-table* from formation level	Minimum CBR (per cent)					
	Non-plastic sand	Sandy clay PI = 10	Sandy clay PI = 20	Silty clay PI = 30	Heavy clay PI ≯ 40	Silt
0.6 m (2ft)	8	5	4	3	2	1
1.0 m (3.3ft)	25	6	5	4	3	2
1.5 m (4.9ft)	25	8	6	5	3	see Note 5
2.0 m (6.5ft)	25	8	7	5	3	
2.5 m (8.2ft)	25	8	8	6	4	
3.0 m (9.8ft)	25	25	8	7	4	
3.5 m (11.5ft)	25	25	8	8	4	
5.0 m (16.4ft)	25	25	8	8	5	
7.0 m (23ft) or more	25	25	8	8	7	

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

Notes:

1. Since the values given in the Table are estimated minimum CBR values, wherever possible the CBR should be measured by laboratory testing at the appropriate moisture content.

2. This Table is to be used only in conjunction with the design chart.

3. With structured clays, such as the red coffee soils of East Africa, laboratory CBR tests should be undertaken whenever possible. Soils of this type can be identified by the fact that their plasticity, as indicated by the Atterberg limits, tends to increase when the soil is worked and its structure is broken down. If CBR tests cannot be undertaken, an approximate estimate of the effective subgrade CBR for this soil type will be obtained by using the values quoted in the Table for sandy clays (PI = 20 per cent).

4. The table cannot be used for soils containing appreciable amounts of mica or organic matter. Such soils can usually be identified visually.

5. Laboratory CBR tests are required for pure silt subgrades with water-tables deeper than 1.0 m (3.3ft).

In Category II where the water-table does not affect the moisture content of the subgrade, the maximum moisture content is unlikely to exceed the optimum moisture content for compaction by the BS light method of compaction provided that there is little likelihood of flooding.

Subgrades coming under Category III will have a moisture content that is virtually the same as the uncovered soil at the same depth and the CBR should be measured at that moisture content.

All of these methods of estimating the subgrade moisture content assume that the water will not penetrate to the subgrade through the base and sub-base. If permeable base and sub-base materials are used in the conditions covered by Categories I and II, the subgrade CBR should be determined on saturated specimens.

The philosophy of pavement design is that the pavement should be made up of layers of material which increase in strength from the bottom to the top of the structure. A large difference between the strengths of two adjoining layers is undesirable because high tensile stresses develop when this occurs.

If the soil has a CBR of less than 5 per cent, it is French practice to place a 15 cm layer of stronger soil at the top of the subgrade to exclude the possibility of having a relatively strong sub-base material lying directly on a very weak subgrade(8). On the other hand, more detailed design curves(7) allow the engineer to use his discretion in selecting the most economical combination of local materials for the layers more than 100 mm below the bottom of the base.

It should be noted that the density at which the CBR of the subgrade is measured can vary from country to country. For example, the French authorities specify a density of 95 per cent of the density obtained from the Modified Proctor compaction test(8). British practice is to test the material in the laboratory under the same conditions of density and moisture content that can be obtained in the field(7). With normal compaction equipment a density of 100 per cent of the density obtained from the British Standard Compaction test, 2.5 kg (5.5 lb) rammer method, can readily be obtained in the field.

Where possible, active material such as expansive clays should not be used as fill in the formation or in embankments. It is recommended(8) that the liquid limit of these materials should not exceed 70 per cent, the plasticity index should not exceed 40 per cent and the linear swell measured in a CBR mould should not be greater than 3 per cent. Whenever it is possible, material with a much lower plasticity should be used.

The sub-base

Many natural materials such as lateritic and quartzitic gravels, partly decomposed rock, river gravels, sand-clays, corals etc can be used successfully in the sub-base of bitumen-surfaced roads. The main requirement is that the material must have a minimum CBR of 25 or 30 per cent(7,8) depending on the design method that is used and the density at which it is tested. Slight differences in the methods of testing specified by various countries have little effect on the overall strength that is measured. However, once again the moisture content at which the test is carried out can be critical. Unless the sub-base will not be affected by water percolating down through a permeable base, or by capillary wetting from a water-table that is close to the surface, it should be tested in a soaked condition.

Every endeavour should be made to use the cheap local materials for the sub-base before considering the importation of material from some distance or the treatment of sub-standard local materials.

A list has been compiled(8) of materials that are often used in the sub-bases of roads carrying various categories of traffic and it is reproduced in Table 7. Some materials such as sandy-gravels can be excellent sub-base materials, irrespective of the volume of commercial traffic that is expected to use the road.

TABLE 7

Suitable sub-base materials

Traffic	Sub-base
T_1 100-300 vehicles/day	natural lateritic gravel argillaceous sand, improved by on-site grading slag and volcanic lava ash and pozzolanas shell deposits sandy gravel crusher-run material 0/60 mm
T_2 300-1000 vehicles/day	natural lateritic gravel (treated as necessary) slag and lava shell deposits treated with bitumen soil-bitumen soil-lime or soil-cement crusher-run material 0/60 mm
T_3 1000-3000 vehicles/day	good quality lateritic gravel (treated as necessary) soil-bitumen (from mixing plant) slag and treated lava treated shell deposits crusher-run material 0/60 mm soil-lime or soil-cement
T_4 3000-6000 vehicles/day	best quality lateritic gravel (preferably treated) crusher-run material 0/60 mm soil-bitumen from mixing plant soil-lime or soil-cement (plant mix)

The Base

Bases have been constructed from a number of materials and these will be considered under the following groups:

I. Natural materials

II. Crushed stone

III. Stabilized materials

Natural materials

The natural materials listed as being suitable for sub-bases should also be examined when the base material is being selected. For bases under bituminous surfacings they should have a grading that is mechanically stable and should contain sufficient fines to make the base dense and to reduce its permeability. The grading limits that are specified by authorities are similar and are all based on the ideal theoretical grading which gives the maximum density. Typical particle-size limits are reproduced in Table 8(1).

TABLE 8

Limits of particle-size distribution for base materials

BS sieve size	Percentage passing				
	Nominal maximum size				
	75 mm (3in)	38 mm (1½in)	19 mm (¾in)	9.5 mm (⅜in)	4.75 mm (3/16in)
75 mm (3in)	100	-	-	-	-
38 mm (1½in)	80-100	100	-	-	-
19 mm (¾in)	60-80	80-100	100	-	-
9.5 mm (⅜in)	30-65	40-75	80-100	100	-
4.75 mm (3/16in)	25-55	30-60	50-85	80-100	100
2.36 mm (No 7)	20-45	25-50	35-70	50-80	80-100
425 μm (No 36)	10-30	15-30	15-35	25-50	25-55
75 μm (No 200)	5-15	5-15	5-15	10-25	10-25

The plasticity requirements for these materials vary depending on the climatic conditions in the particular region. However, because of the build up in moisture beneath a sealed surface, the plasticity of these bases must be controlled much more rigidly than is necessary for gravel-surfaced roads. In moist or wet tropical regions, the liquid limit, plasticity index and linear shrinkage of material passing the 425 μm sieve should be less than 25 per cent, 6 per cent and 4 per cent respectively. Preferably it should be non-plastic. In dry areas more plastic materials may make excellent bases but care should be taken to ensure that these bases will not become too moist.

When local gravels and laterites do not exactly meet all of the requirements of the specification it is possible sometimes to produce an acceptable material by crushing, screening or blending with other materials. This should not be overlooked, especially where satisfactory untreated material must be hauled long distances to the site.

Crushed stone

There are two methods for building bases with crushed rock. Waterbound macadam is constructed from layers of single-sized crushed stone into which sand fines are watered and rolled. If the layers are too thick, it is difficult to fill the voids in the rock with sand and, therefore, it is usual to limit the layer thickness to twice the maximum size of the stone. The maximum size of the fines, which must be non-plastic, should be 5 mm.

A graded, crushed stone that conforms to the particle size requirements of Table 8 can be compacted into a dense material with a low volume of voids. The material has a tendency to segregate and often there is only a narrow range of moisture contents within which it can be compacted into a good quality, dense base. Hence, close supervision of the construction is required to ensure that a high quality product is obtained.

The stone should be sufficiently strong and durable to prevent its breakdown and deterioration under traffic. Partly weathered igneous rock may appear to be sound but it can contain minerals that are already decomposed and the combined action of traffic and moisture will cause rapid failure.

A wide range of rock types can be used successfully but limestone is generally one of the most satisfactory. Both natural gravels and crushed rock should have a minimum CBR after soaking of 80 per cent.

Stabilized bases

Where suitable base materials are not available within an economical haulage distance, local natural materials can be stabilized to provide high quality bases for roads with bituminous surfacings.

In moist and wet tropical areas, many of the natural gravels are too plastic to use, without treatment, as a base under a sealed surface. Lime and cement react very well with almost all of these gravels and both have been used successfully as stabilizing agents in Africa. In the dry areas where there are cohesionless sandy soils, cut-back bitumens can be used successfully as a stabilizer but if the cost of bitumen is high, cement stabilization is an alternative. Lime requires some plastic material in the soil with which it reacts to form cementitious material and, therefore, is unsuitable for stabilizing cohesionless soils.

Cement has been used more widely as a stabilizing agent in Africa than has lime(11). The use of bitumen has been limited to areas where it is readily available and the soils are amenable to this form of stabilization.

The preference for cement compared with lime is difficult to justify because the construction procedure for lime stabilization is much less critical and, with plastic materials, lime can react well to form a material of the same quality as soil-cement. This preference is caused partly by the additional publicity that soil-cement has received, but the difficulty which can be experienced in obtaining sufficient lime for a project and its artificially high cost in relation to cement are contributing factors.

For many centuries, lime has been produced in simple kilns on a village industry basis in parts of Africa, the Middle East and Asia. Although the quality of this lime does not compare with the commercial product, it is quite satisfactory for the stabilization of plastic gravels and clayey soils. Research is being carried out(13) into the use of locally-burnt lime for the production of lime-stabilized bases and it is hoped that this form of construction will be used more frequently in the future.

The amount of cement or lime that is required by a particular soil must be determined from laboratory tests. Many different criteria have been used throughout the world but experience in Africa indicates that the most satisfactory design criterion for this continent is based on the CBR test(11). The French authorities recommend that the laboratory CBR of the mixture when compacted to 100% of the Modified Proctor Density should be 160 to 200 per cent after 3 days of curing and 4 days of soaking. On the other hand, Britain recommends that the CBR should be at least 100 per cent when compacted to the density that is expected in the field and subjected to the same conditions of curing and soaking. The apparent anomaly can be explained in terms of the difference in the degree of compaction at which the test is conducted. Unconfined compressive strengths of 15 and 20 bars after 7 days are also recommended by the French(8, 12) as being indications of the suitability of stabilized materials for road bases. However, many engineers prefer the minimum strength to be 25 bars.

Mixtures of bitumen and sand have not received the same detailed study as mixtures of soil with cement or lime and there is come uncertainty about the most suitable criteria for designing mixtures. The Hubbard-Field test is probably the most commonly used with unsoaked stabilities of the order of 500 to 700 kg at 60°C being specified(8, 12) but it has been suggested that lower stabilities may be satisfactory(11).

The great majority of stabilized soil roads in Africa have bases which are from 10 to 15 cm thick overlying a sub-base, usually of untreated gravel. However, there is a real danger of failure when a thin layer of a stiff, stabilized material is placed over a sub-base with a CBR of 25 per cent. One passage of a heavy load can cause excessive tensile stresses in the stabilized base, which result in its ultimate failure. To help prevent this form of failure, it is recommended that the minimum thickness of stabilized bases should be 15 cm.

There are two basic methods by which stabilized layers can be mixed and placed, viz mix-in-place and plant-mix. Many machines for applying the mix-in-place technique have been tried with varying degrees of success. One of the limiting factors is the ability of the machine to break up the in-situ soil and blend the stabilizer into it so that the product is a fine tilth. The plant that has been used for the stabilization of road bases in Africa has been split into five categories and the soil types for which these categories are suitable are listed in Table 9(11).

TABLE 9

Soil plasticity limits for stabilization using different types of plant

Type of plant	Plasticity index of the soil multiplied by the percentage finer than 425 μm (no 36 sieve)	Normal max depth capable of being processed in one layer - cm
Agricultural disc harrows disc ploughs etc and motor graders	$\ngtr$ 1000	12 to 15
Light rotavators (<100 hp)	$\ngtr$ 2000	15
Heavy duty rotavators (>100 hp)	$\ngtr$ 3500	20 to 30 depending on soil type and horsepower available
Single pass stabilisers	$\ngtr$ 2000 to 3000 depending on the horsepower	20
Static mix plant	$\ngtr$ 500	no limit

When lime is used as the stabilizer, the time during which the mixing and compaction must be completed is not critical and all of the categories of mixers can be used satisfactorily provided that due allowance is made for the uneven mixing produced by categories 1 and 2. The mixing and compaction must be completed in the shortest possible time when cement is added and it is recommended that heavy duty rotavators, single-pass stabilizers or static plant-mix methods should be used if possible. Both mechanical spreaders and manual distribution are suitable for spreading cement and lime when mix-in-place methods are used.

Normal rollers and graders should be used for compacting and shaping the base prior to curing it as soon as possible after the completion of the final shaping. There is some doubt about the advisability of disturbing the compacted surface during the final shaping and the disturbance during this operation should be kept to a minimum. Curing can be carried out by regularly spraying with water, by covering it with soil or straw and keeping it damp, or by spraying a bituminous curing membrance on the surface. Unless the curing is carried out properly, a significant percentage of the water in the compacted material will dry out, undesirable cracking will occur in the surface of the layer and the strength of the base will be reduced.

There has been a number of failures of pavements with stabilized bases, particularly when cement has been used as the additive. Most of the failures can be attributed to one or more of the following causes:

I. Inappropriate strength criteria for the material.

II. Inadequate construction thickness of the stabilized layer.

III. Poor construction control or curing.

If the design and construction is carried out carefully, stabilized bases will give excellent service, particularly in the African climate where frost penetration is not a problem.

A list of materials which are suitable for use in bases is included in Table 10.

TABLE 10

Suitable base materials

Traffic	Base
T_1 100-300 vehicles/day	lateritic gravel, natural or treated (with cement, or lime) soil-bitumen (plant mix) sandy gravel soil-lime or soil-cement slag and selected lava shell deposits treated with bitumen crusher-run material 0/40 mm

(TABLE 10 (Continued))

Traffic	Base
T_2 300-1000 vehicles/day	best quality lateritic gravel (treated as necessary) high quality sandy gravel soil-bitumen (from mixing plant) or soil-cement slag and treated lava crusher-run material 0/40 mm
T_3 1000-3000 vehicles/day	best quality lateritic gravel (preferably treated) high quality sandy gravel crusher-run material 0/40 mm
T_4 3000-6000 vehicles/day	lateritic gravel, or sandy gravel, treated in mixing plant crusher-run material 0/40 (preferably treated with cement or bitumen) gravel-bitumen or gravel-cement

Pavement Design

The drainage of roads with impermeable layers presents no problem, because water cannot penetrate through the layers and weaken the underlying material. Provided that the cross-falls are adequate, the bituminous surfacing is kept in good repair and the shoulders are well maintained, surface water will drain away quickly without doing any damage. A suitable cross-section is shown in Fig 4.

If the bituminous surfacing over a permeable base fails and allows water to enter the base, the road must be shaped so that this water drains out of the base as quickly as possible. A cross-section of the type shown in Fig 5 should be used. Because the base and sub-base are extended to the drainage ditches, the water cannot collect at the edge of the carriageway and reduce the strength of the base at this critical point. If it is too costly to extend the base and sub-base as indicated, drainage trenches must be cut through the shoulder from the edge of the base to the ditch at spacings of 3 to 5 m. It is necessary that the bottom of the trenches should be lower than the bottom of the sub-base. Under no circumstances should the pavement be constructed inside a "box" so that it is confined within continuous impervious shoulders.

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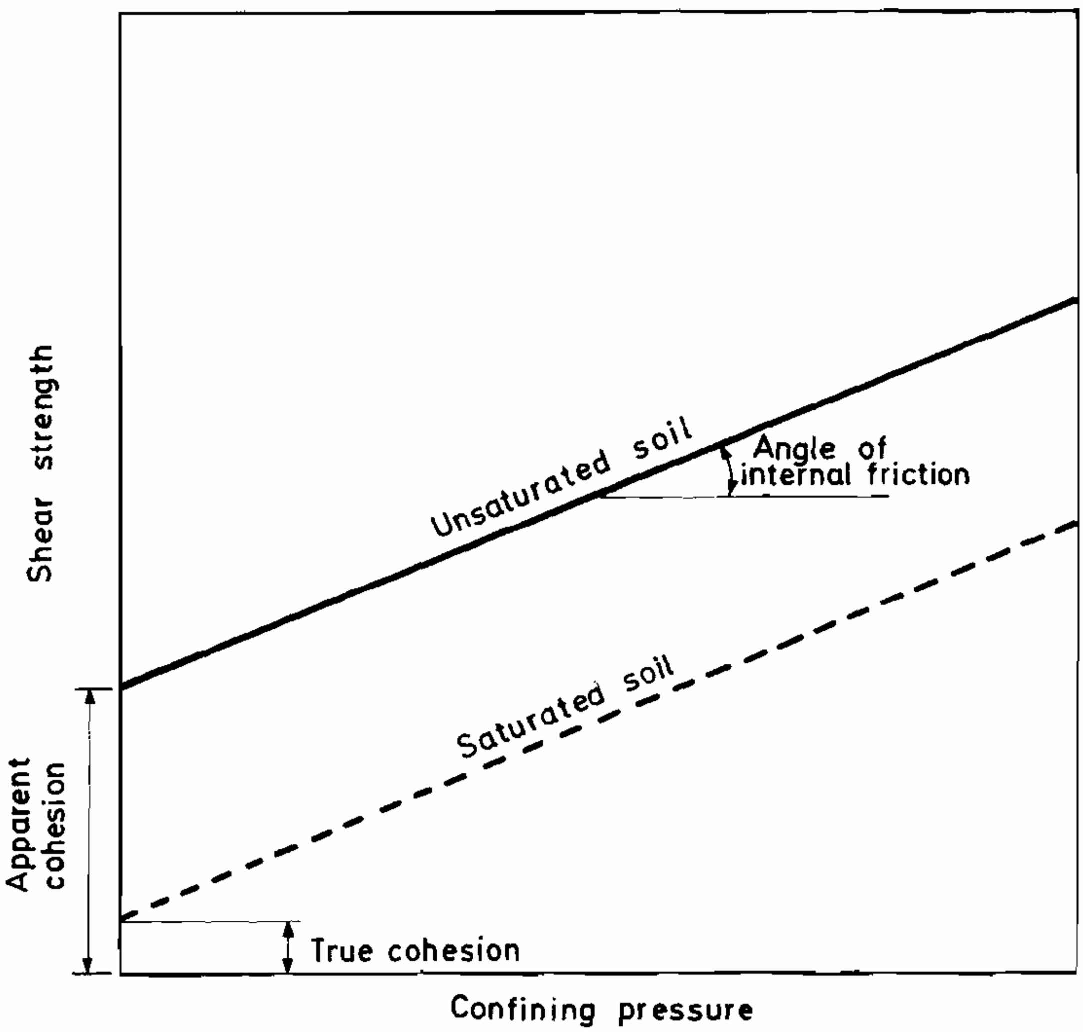


Fig.1 THE COMPONENTS OF THE SHEAR STRENGTH OF SOILS

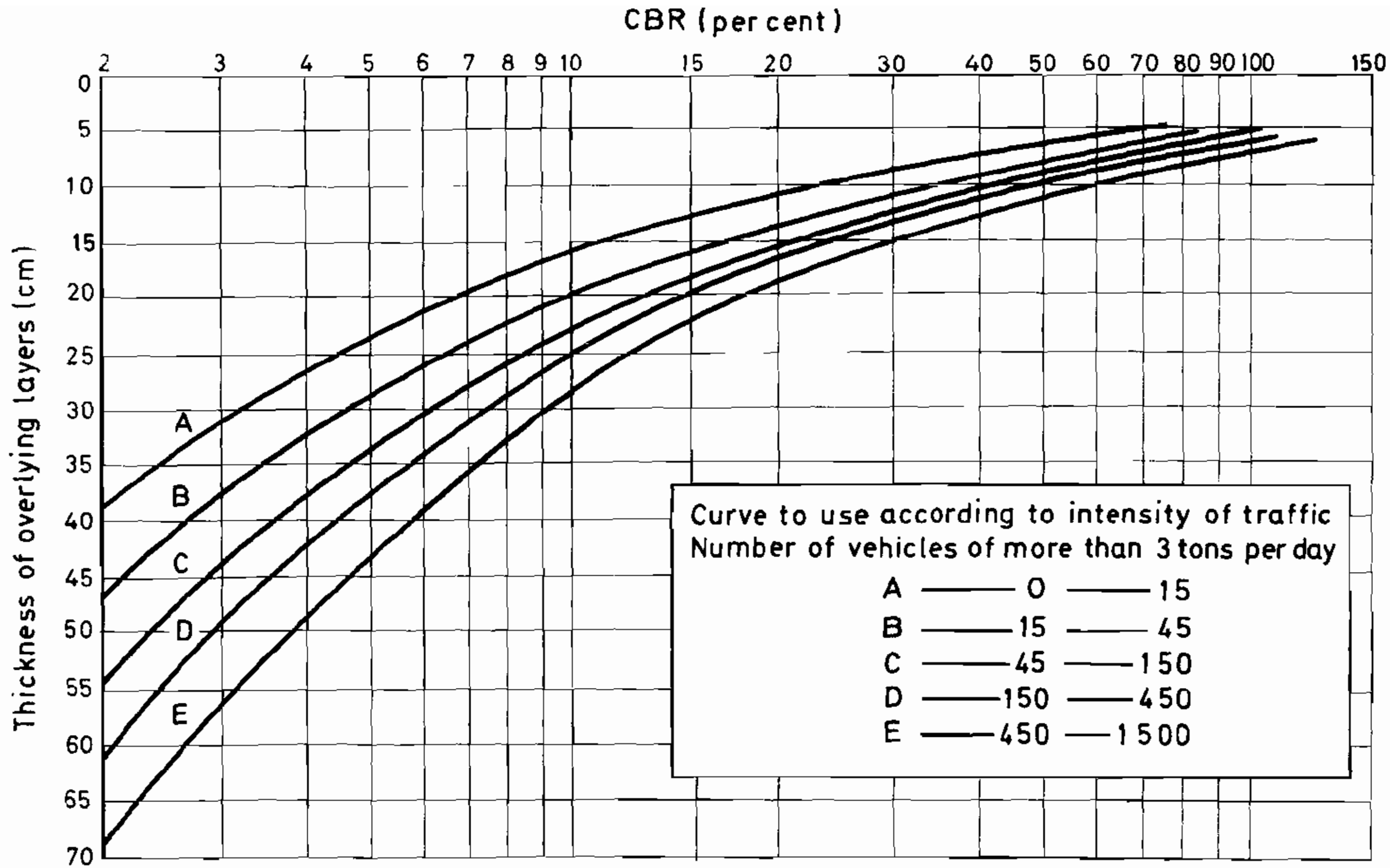
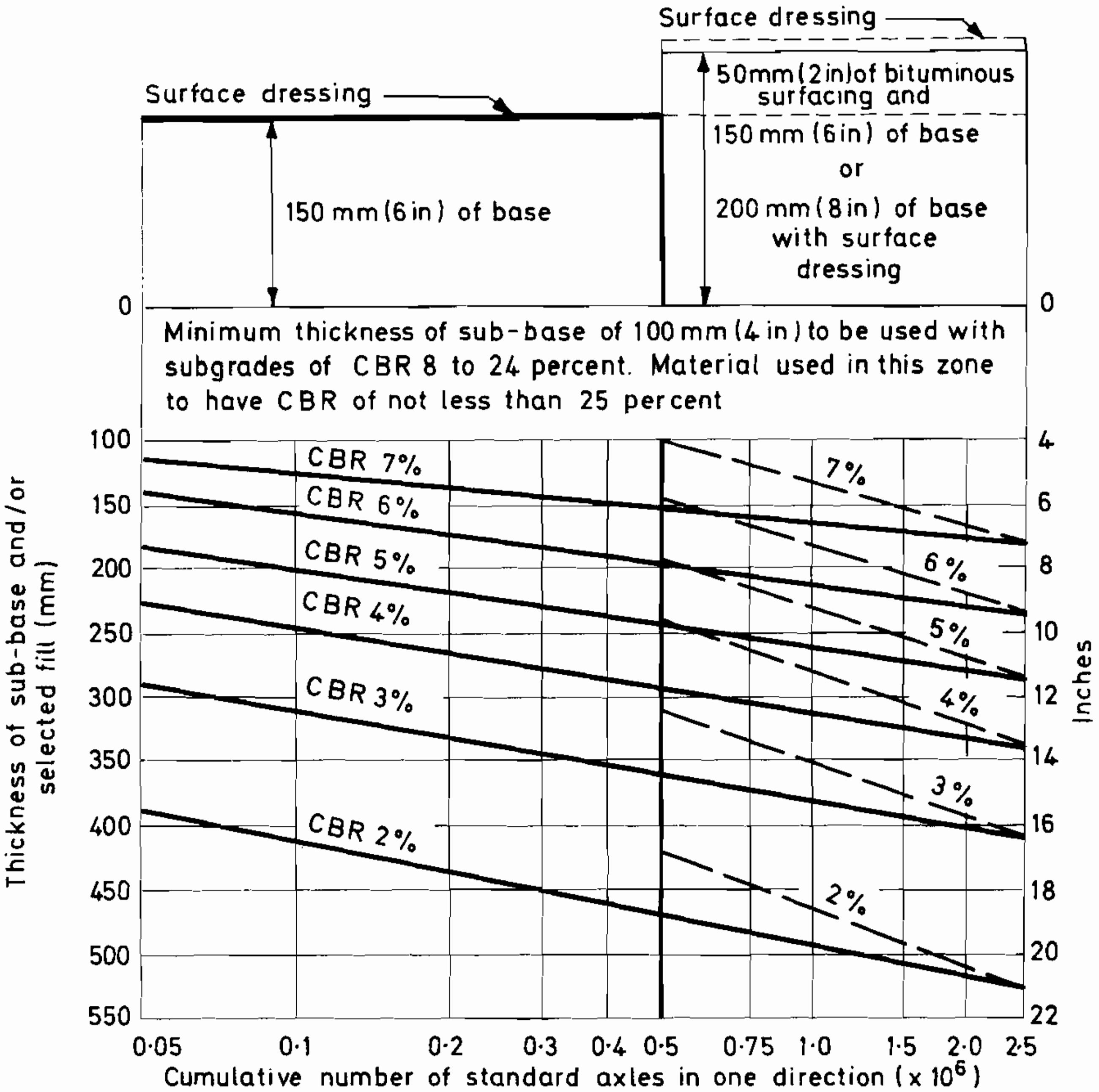


Fig. 2 PAVEMENT DESIGN CHART FOR UNSURFACED PAVEMENTS



If it is desired to provide at the time of construction a pavement capable of carrying more than 0·5 million standard axles the designer may choose either a 150 mm (6in) base with a 50 mm (2in) bituminous surfacing or a 200 mm (8 in) base with a double surface dressing. For both of these alternatives, the recommended sub-base thickness is indicated by the broken line.

Alternatively, a base 150 mm (6 in) thick with double surface dressing may be laid initially and the thickness increased when 0·5 million standard axles have been carried. The extra thickness may consist of 50 mm (2 in) of bituminous surfacing or at least 75 mm (3 in) of crushed stone with double surface dressing. The largest aggregate size in the crushed stone must not exceed 19 mm (3/4 in) and the old surface must be prepared by scarifying to a depth of 50 mm (2 in). For this stage construction procedure the recommended thickness of sub-base is indicated by the solid line.

Fig. 3. PAVEMENT DESIGN CHART FOR FLEXIBLE PAVEMENTS

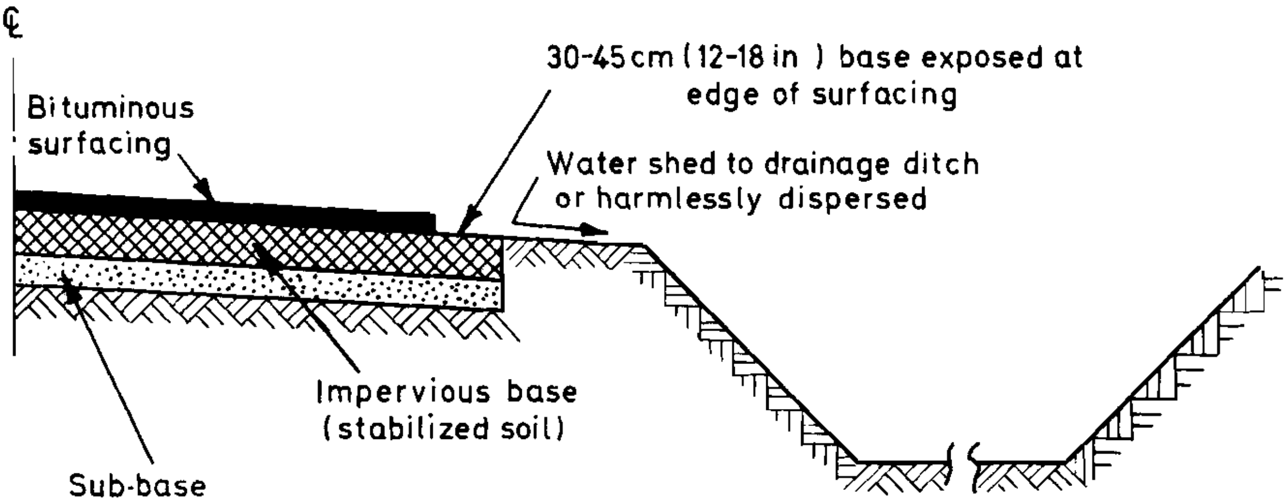


Fig.4 DRAINAGE OF PAVEMENT LAYERS WHEN THE BASE IS IMPERVIOUS

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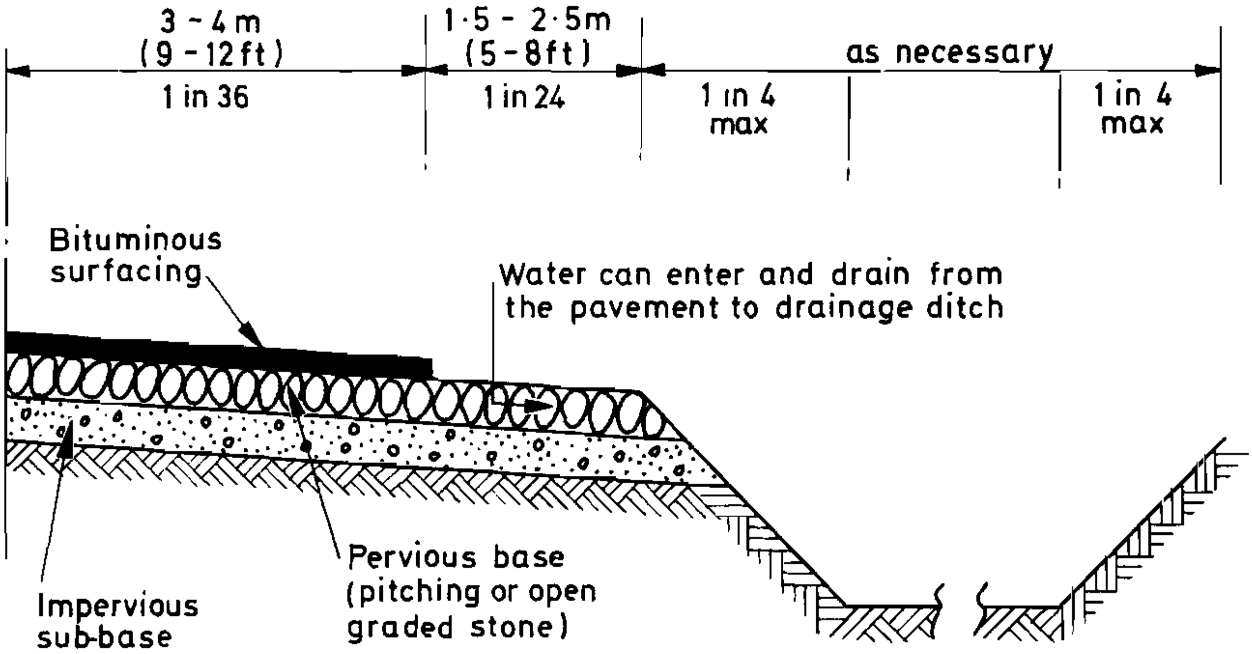

Fig. 5 DRAINAGE OF PAVEMENT LAYERS WHEN THE BASE IS PERVIOUS

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