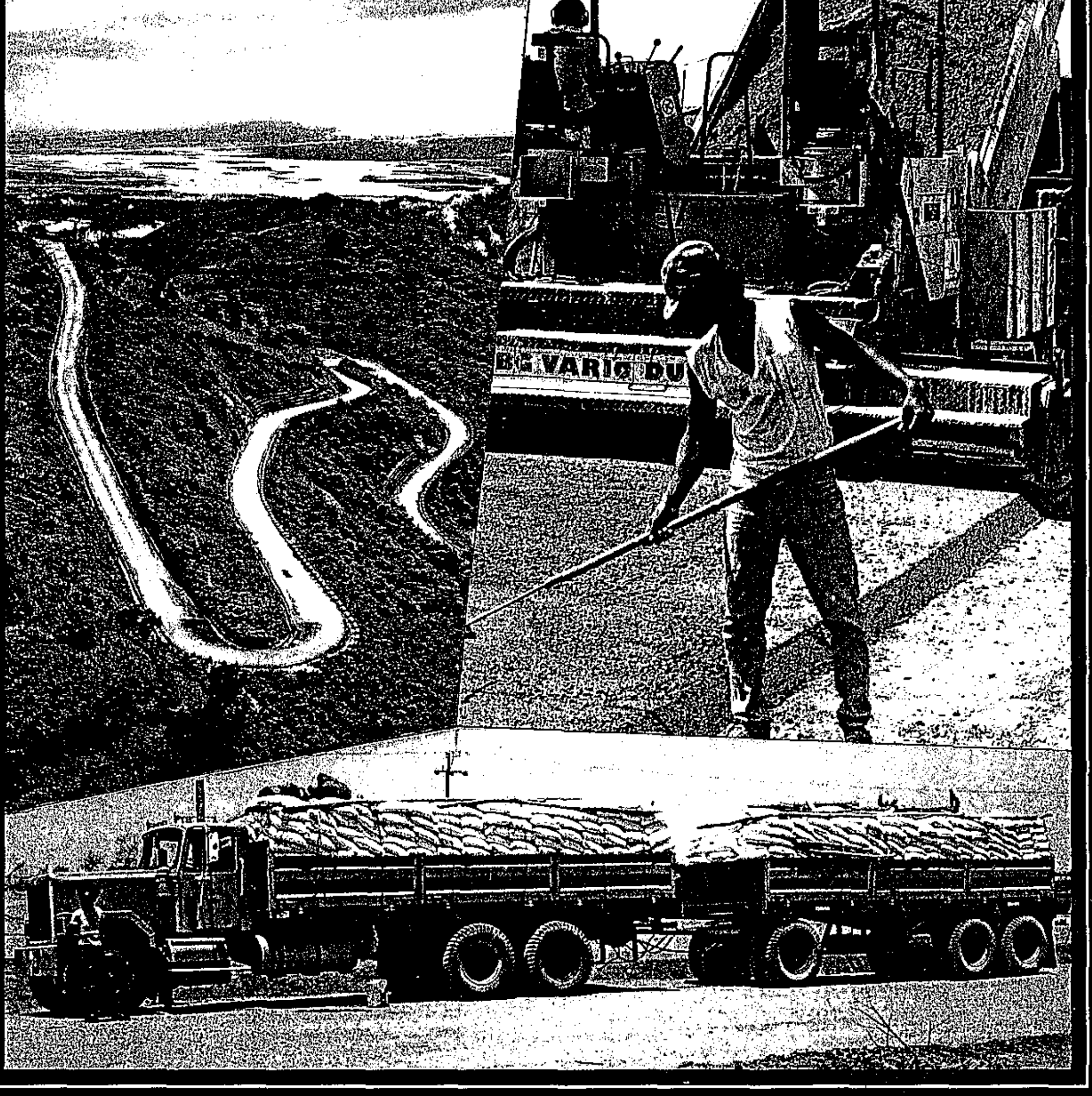
State of the Art Review / 9

Road Building in the Tropics



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STATE-OF-THE-ART REVIEW 9

ROAD BUILDING IN THE TROPICS

by R S Millard

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Preface

This book is intended for engineering students and for highway engineers practising in hot countries. The soils and other road making materials that are found in the tropics can have markedly different properties from those occurring in more temperate zones. Climatic conditions have a profound effect on both the design and construction of roads and often the presence of grossly overloaded vehicles in the traffic stream produces additional complications. Sophisticated machinery is available for many road building processes and new machines and processes are constantly appearing on the market. There are often pressures to use financial aid programmes in acquiring road building machinery that may not be appropriate for local conditions.

The book describes how all these factors affect the design, construction and maintenance of roads. The intention is to impart an understanding of why road pavements behave as they do, so that highway engineers can make the many judgements that are necessary in providing the most effective road system for the local conditions.

It is neither possible nor desirable to provide comprehensive descriptions of all the techniques involved in road making. The essential features of materials testing and of road design, construction and maintenance are described and references are given at the end of each chapter as to where detailed information on the techniques can be found.

Many of the diagrams and tables have been reproduced from other publications. A particular debt is owed to the British Standards Institution and to the American Society for Testing Materials in this respect. Assistance has also come from many other individuals and organisations. Amongst these, my ex-colleagues of the Overseas Centre of what is now the Transport Research Laboratory merit special mention and I gladly record my sense of gratitude to them all.

Contents

1 The context

Amongst the great changes that have been brought by the twentieth century one of the most remarkable lies in communications. The nineteenth century had brought steam power and, with it, an advance in our ability to travel by land and by sea. With this century has come the internal combustion engine and radio communications, bringing an even greater advance in the ease with which ideas, people and goods can travel around the world. There is the South Carolina farmer with his sophisticated cultivation and marketing methods and there is the Senegalese peasant with his hoe, both of them growing ground nuts. Fifty years ago they lived in different worlds. Now they are less than seven hours travel apart and each has an awareness of the vast differences between their methods of production and their life styles. We are still learning to cope with this situation. Only in the last decade have we come to realise that quite drastic actions will be needed to make sure that the forces that have been released really do fulfil their potential to promote the welfare of mankind.

There is concern about tropical rainforests, about atmospheric pollution and, perhaps less immediately urgent, about our limited store of fossil fuels. On these subjects our ideas have yet to be clarified and there will be no more mention of them. They are mentioned here only because the concept of making the best use of the world's resources in building and maintaining our roads provides a main motivation for this book.

The other motivation lies in the very large differences between conditions in the industrialised countries and the rest of the world. There are three ways in which these differences affect road engineering. One lies in the state of economic and social development reached when the era of the motor vehicle arrived. Another lies in the nature of the available road making materials i.e. the soils and the rocks. The third derives from differences in climate.

Most European countries started the era with quite dense road networks. Some parts of these followed the lines of the military roads built by the Romans, but most were created, along with the railways and canals, in response to the hugely increased demand for transport following the industrial revolution.

In proportion to the total lengths of the networks, the amount of new road building has not been particularly large. Much of this new road building has been to provide systems of inter-urban motorways. At the other extreme are the many roads on the housing estates built to serve the needs of expanding urban populations. In between are the bypasses built to relieve the road traffic congestion that developed in many towns and villages. Alongside this new road building was another activity, now almost forgotten by the present generation, namely the modernising of the existing rural and urban road network, strengthening and often widening the roads and providing them with all-weather surfacings. Now, all or nearly all the networks of public roads in most West European countries have running surfaces of asphalt or concrete. New road building still continues and, alongside this, the major task lies in the maintenance and improvements of all the roads and the bridges to carry the still increasing road traffic.

In response to these demands, expertise has been well developed in the upgrading of existing roads and in the strengthening and maintenance of pavements. Skilled consultants and contractors are available, eager to sell their services for all the tasks of road making. In France, for instance, the 1980's saw road contractors seeking to promote many innovations, each firm attempting to produce more effective solutions than its rivals. In 1985 there were 76 products or processes under review as possibilities for improving the performance of surface dressings together with no less than 197 products or processes under consideration for asphalt mixtures.

In North America there was a different situation. The coming of the motor vehicle found the streets in most of the cities already fairly well paved, but between the cities there were immense distances over which the roads, if they existed at all, were often little more than gravel tracks, and the local rural road networks were often even more rudimentary. The Americans responded with typical energy and purpose. One outcome was the construction of the Inter-State Highway system started in the late 1920's and now substantially completed. Indeed, on the eastern and western seaboards of the USA, they are well into the second generation of Inter-State Highways, built in new locations to avoid the areas of concentrated development that grew rapidly along the earlier Inter-State Highways. The paving of secondary and minor roads proceeded more slowly and there is still a considerable length of rural roads surfaced with gravel and broken stone. There are two features which mark the American road building scene. One is the emphasis on speed of construction, resulting in the development of larger and larger machines for earth moving and for pavement construction; the larger slip-form pavers are capable of laying concrete pavements at a rate approaching one kilometre per day. The other lies in the contribution made in the early days of major road and airfield construction to the development of testing methods for soils and other materials used in pavements. Many of these, the Standard Compaction Tests and the California Bearing Ratio Test for instance, have been adopted for use all over the world. And in the 1930's, those pioneers of the science of soil mechanics, Casagrande and Terzaghi, were working in the USA to establish an understanding of the behaviour of soils as foundations for civil engineering structures and their work provides a basis for much of our knowledge today.

This was a period when many ex-colonial countries in Africa and Asia were moving towards independence. They, and some South American countries, had extensive railway systems. In the larger countries, India for example, these remain an important component of the land transport system. Some also had the rudiments of a main road system. In West Malaysia and Sri Lanka there were main road networks already paved with the bitumen-grouted macadam that was the common road building method in Europe before the second World War. In East Africa the bituminisation of the road from Mombasa to Lake Victoria had begun and there were similar beginnings elsewhere in tropical and sub-tropical regions. Whilst the development of road transport in industrial countries was well under way by the middle of the century, it really began elsewhere in the world after the second World War. In the ex-colonial countries it was given a large boost by the coming of independence, as is shown in Table 1.1. The numbers of road vehicles increased between ten- and twenty-fold in the years between 1950 and the late 1980's and, although the growth rate has slackened since then, it is still greater than in the industrialised countries. At the beginning of this period some countries had a basic main road system, but few had the secondary roads and the farm-to-market roads necessary to spread the benefits of a cash economy and to build up the social services, including education and medicine. Even today, less than half of the rural population of India is within 5 kilometres of a motorable road.

TABLE 1.1
BASIC ROAD STATISTICS 1930-1990

Country and year	Length of public road *Thousands of km*	Proportion with permanent surface *Percentage*	No. of motor vehicles *Thousands*	Vehicles per thousand population
Argentina				
1930	80		25	2
1950	170	10	490	25
1970	319	17	233	92
1986	337	27	5233	172
Kenya				
1930	<4	0	<4	<1
1950	21	2	14	2.0
1970	70	7	114	10
1984	87	12	219	11.5
Malaysia				
1930	<4	2	<10	<1
1950	9	55	91	6
1970	28	85	312	16
1988	64	81	2681	91
UK				
1930	252	77	2274	44
1950	294	96	4010	80
1970	534	100	13,788	244
1988	563	100	21,163	372
USA				
1930	1700	20	11,600	110
1950	2406	29	34,852	230
1970	6003	44	109,304	529
1987	6233	56	179,044	561

By the late 1950's, International Aid Agencies were coming to grips with the problem of development in the poorer countries of the world. The World Bank, originally conceived as a means of restoring war-torn Europe, had become a major agency for this purpose, the Regional Economic Commissions of the United Nations were active in providing financial and technical assistance and most of the industrialised countries had their own programmes of assistance. A large proportion of this aid was devoted to building up infrastructure, providing facilities for power supply, water supply, transport and communications, etc. In the transport sector the most insistent demand was for more roads to build up the internal communication systems so that the commercial, political and social life of the countries could prosper.

Things went well for a while. The initial effort was directed towards the main road systems. To this was added the provision of roads as part of regional and local rural development schemes and roads to cope with the traffic problems of the rapidly growing urban areas. One of the original premises of this financial assistance was that it should be confined to capital works, with the presumption that increasing prosperity would make it possible for the recipient countries to fund and to undertake all the necessary maintenance required on the new facilities. This was all very logical, in financial terms at least. It was quite right that a developing country should learn to manage its own financial affairs so that the resources needed for annual recurrent expenditures could be found from government revenues. But with roads this did not work. Governments had difficulties in raising the necessary revenues and many of them were quite unable to build up technical organisations with the skills necessary for effective road maintenance. Major new construction had been managed quite well, often with increasing local participation in the activities of the consultants and contractors who had come from abroad to design and construct the new roads. In contrast, efforts to establish effective road maintenance have failed miserably in almost every country. Many of the main roads which were built in the early days of international aid have broken up, sometimes because traffic loads have been heavier than were expected, but more commonly because the routine and periodic maintenance necessary to preserve them has not been done. It has also proved impossible, in Africa particularly, to establish the local road maintenance organisations needed to cope with road maintenance. Table 1.2, taken from a recent World Bank report, summarises the current condition of main roads in the countries of the six World Bank Regions and there is no doubt that the situation has deteriorated further in some countries since these data were collected.

What has gone wrong and what can be done to put the matter right? The World Bank report 'Road Deterioration in Developing Countries', provides a detailed analysis of the situation and contains

TABLE 1.2
CONDITION OF MAIN ROADS (PERCENTAGES WEIGHTED BY LENGTH OF COUNTRY NETWORKS)[a]

Region	Bituminous or concrete surface			Gravel or earth surface		
	Good	*Fair*	*Poor*	*Good*	*Fair*	*Poor*
Eastern Africa	42	32	26	42	30	28
Western Africa	52	23	25	20	36	44
East Asia	20	59	21	41	34	25
South Asia	19	45	36	6	39	55
Europe, Middle East, and North Africa[d]	41	35	24	30	46	24
Latin America and Caribbean	44	32	24	24	43	33
Average	32	42	26	31	36	33
United States (Federal Aid Network, 1981)[b]	31	57	12	-	-	-
United Kingdom (Trunk Road System, 1983)[c]	85	12	3	-	-	-

a As reported in an internal World Bank survey of 85 countries based, as far as possible, on published pavement condition information (60 countries) and supplemented, where necessary, by the judgement of Bank highway engineers.

b After *Public Works Infrastructure: Policy considerations for the 1980's*. Budget Office, US Congress (1983).

c Adapted from *Investment in the Public Sector Built Infrastructure - Report A: Roads and Bridges* UK National Development Office (1985).

d Europe in this context means those European countries in receipt of assistance from the World Bank

Source: World Bank Report No 6968 (1987), 'Road Deterioration in Developing Countries' a comprehensive review of what can be done to get back on course in building up effective highway systems in all these countries. For some time aid agencies have recognised the need to provide financial and technical help in promoting effective road maintenance, and governments are making determined efforts to accept the costs of such maintenance as a proper charge on their revenues. Priorities vary from country to country but there are two common themes that merit mention here. One is that government highway departments have become heavily motivated towards new construction. Most of the bright engineers gravitate towards the construction departments because of the technical interest and the financial rewards that such work provides. The other is that, in most countries, the secondary and minor road systems are not in a condition for ordinary road maintenance procedures to be at all effective. It can be concluded that there is an urgent need to develop local road rehabilitation industries. In such industries, highway officials and contractors share a knowledge of how to use locally available road making materials in road rehabilitation processes that are the most effective in the local environment. With such an industry in place, a country will have the means available to continue with the development and maintenance of its road systems by highly motivated people in government and in the commercial sector.

The locally available road making materials, the soils and the rocks, vary considerably from country to country and the road making techniques must be varied accordingly. The soil mantle that covers the rock over most of the earth's land surface is derived from the weathering of rocks. Nature's engines for producing this weathering are fuelled by heat and water in combination, and it follows that this weathering is most active in tropical climates. Most of the soil cover in more temperate and colder regions is either the remnant from geological times when warmer climates prevailed, or has been brought in from elsewhere by wind and water and by glacial action during past ice ages. The soils and rocks have evolved to a relatively stable condition and it is usually not difficult to define their characteristics for use in road engineering.

Things are different in the tropics. Soil forming processes are very active and the surface rocks are usually deeply weathered. Most of the area of the tropics is covered with residual soils deriving from the rocks in the immediate vicinity, and many of these residual soils are still undergoing chemical and physical alteration. The intense leaching of the soil in hot humid climates produces hill slopes that can be very unstable and soils that are susceptible to erosion. Concentrations of particular soils occur, for instance the black cotton soils which contain the active clay mineral montmorillonite, and the laterites in which iron oxides have accumulated. In volcanic areas the weathering of volcanic debris produces soils with unusual and often rather transient properties. Even the transported soils e.g. the desert sands and the alluvial silty clays of the river deltas, are usually of fairly recent origin, lacking the stability that geological age can bring.

Standard methods of testing have been evolved which are now in use all over the world. But it has only recently been realised that these testing methods often need to be modified in order to reveal the nature and the likely engineering behaviour of many tropical soils. In addition, the interpretation of the test results has been primarily based on experience gained in temperate climates and this interpretation needs to be modified so that it copes adequately with the wide variations that occur in climate in other parts of the world.

Intense heat is one of the characteristics shared by most tropical countries but there are wide variations in the extent of diurnal and seasonal temperature changes. In some countries, southern

Thailand and Malaysia for instance, air temperatures vary by little more than 5°C, whilst in other places, the Arabian Gulf for example, air temperatures may change by over 40°C between day and night with an even larger annual range. Moisture conditions also vary enormously, ranging from perpetually arid, through monsoon climates having one or two wet seasons in the annual climatic cycle, to regions of persistent high humidity. All of these have a large effect on the design of roads, on the practicality of different road construction and maintenance processes, and on the performance and durability of road pavements under traffic.

Vehicle loading is a part of the context. In industrialised countries there are regulations on the size and weight of vehicles. These regulations have two purposes. One is in the interests of road safety. The other is to contain the weight of vehicles within the carrying capacity of the road pavements and bridges. There are minor differences between countries but the trend is towards increasing conformity in the interests of international traffic. These regulations are quite strictly enforced. Some Third World countries have similar regulations concerned with the construction and use of road vehicles but enforcement has usually proved to be quite impracticable. In many countries, vehicles that are grossly overloaded are very much in evidence amongst the traffic. Indeed, in some countries there is a thriving local industry at the ports for fitting newly imported commercial vehicles with wider and higher body frames so that they can carry bulkier and heavier loads than the vehicles' rated capacities. It is ironic that those countries with the greatest need to build up adequate road networks are handicapped by the damage done by overloaded vehicles to their roads. Damage there certainly is. In Kenya, for example, parts of the Nairobi-Mombasa road were rebuilt in the 1960's. The designers made quite accurate estimates of the people and goods that would be carried on the road but, in their design of the pavement, they assumed that the goods would be carried in vehicles which conformed to the country's regulations on vehicle weight and axle loading. A design life of 15 years or more was expected before the pavements were likely to need further strengthening. Four years after the works were completed the pavements were showing signs of distress and, thereafter, deterioration was rapid and complete reconstruction of the upper layers of the pavements became necessary. A census of vehicle loading revealed that many of the trucks had all-up weights and associated axle loads up to 60 per cent higher than those permitted in the regulations. There can be no doubt that this was the primary reason for the early demise of the pavements.

This is a familiar story in many countries. Clearly it was unduly optimistic to believe that vehicle and axle loading regulations could be enforced with the same rigour as in the industrialised countries. The vehicle owners and operators see things their own way. Very much occupied with the here and now, their concern is to carry goods as cheaply as they can. To them it is obviously best to load their vehicles to the limit. Owners of large fleets of vehicles often maintain them reasonably well and seek to avoid gross overloading, but as soon as the vehicles are passed on to a succession of other owners they are likely to be used to the limit of their capacity.

It is not likely that the ability to undertake effective road maintenance will improve very quickly in those countries where it is deficient and it is likely that heavily overloaded vehicles will be using the roads for some years to come. Both factors must be considered in the design of new road pavements and in the rehabilitation of existing defective pavements. The types of construction must be robust, capable of carrying the heavy loads and, as far as is possible, be capable of withstanding some neglect of routine and periodic maintenance. It is hoped that the information in this book will help in achieving these objectives and will encourage the building up of effective road maintenance organisations.

Reference

THE WORLD BANK (1988). Road deterioration in developing countries. Report No 6968. World Bank, Washington, DC.

2 Tropical soils

2.1 Soil classification and testing for engineering purposes

Most of the earth's land surface is covered with a thin mantle of soil derived from the weathering of underlying rocks. Some soils are residual i.e. they derive from the in situ weathering of the rocks which lie below them. Others have been transported by wind, by water, by glaciation and by the vast tectonic movements of the earth's geological history. Soil provides the foundation which supports life on earth. The huge variety in which this life has evolved reflects the wide variations in the nature of these soils and the different climates that prevail over the earth's surface. These soil forming processes are continuing, only slowly in more temperate regions but more rapidly, sometimes with astonishing speed, within the tropical and sub-tropical belts. The civil engineer needs to understand the nature of these soils and the effects of local climate on the support the soils provide as foundations for engineering structures. To the road engineer this understanding is particularly important. His structures lie in a thin ribbon through the countryside and the soils and rocks are their principal components. He needs rapid and reliable means of determining the engineering properties of these materials, their uses and their limitations.

We start with the simple textural classification used in everyday speech; gravel, sand, silt and clay. For engineering purposes these four soil components are distinguished by limits on the particle sizes they contain.

Gravel	Over 2 millimetres particle size diameter
Sand	2 - 0.06 millimetres particle size diameter
Silt	0.06 - 0.002 millimetres particle size diameter
Clay	Less than 0.002 millimetres particle size diameter

Gravels usually derive from shattered rocks which have been transported by water. The particles are smooth and rounded and are usually very hard, the softer part of the rock having been worn away by attrition. Other gravelly soils are found in many tropical countries e.g. the lateritic gravels. These are soils in which iron oxides, transported in solution, have been redeposited to form gravelly nodules. There are other soils of gravel size which can find good use in road making in the areas where they occur, volcanic cinders for instance and the concretionary calcretes and silcretes found in some desert areas.

Most sands are transported soils. Most frequently their particles consist of silica i.e. quartz, which is amongst the minerals least susceptible to weathering. Some sands consist of calcium or magnesium carbonate e.g. coral sands and sands derived from the coastal erosion of limestones. Some sandy soils are the products of in situ weathering of granitic rocks e.g. the sand-veldts of Zimbabwe, but almost all sands have been produced from weathered rock which has been transported by wind or water.

Silts also consist of finely divided rock. They sometimes occur in the vertical profile of decomposing rocks i.e. as residual soils, but by far their most common mode of occurrence is as soils transported by wind or water. They are often very fertile, as exemplified by the wind-blown loess soils of northern latitudes and the water-borne soils of the Nile delta. But they are not easy soils for the engineer. They are often only weakly consolidated, they lack the free draining characteristics of gravels and sands and, in colder climates, are highly susceptible to frost heave. Soils with a high silt content are particularly susceptible to erosion.

Gravels, sands and silts all consist of rounded or sub-angular particles of unweathered minerals. Clays are altogether of a different nature. They are the products of the chemical alteration of rock minerals by weathering. During this weathering process the rock minerals are recomposed as hydrated alumino-silicates. It is said that the final product of this weathering is kaolinite. Certainly this is the most common form of clay mineral and it is by the properties of kaolinite that the characteristics of clays are generally recognised. Kaolinitic clay consists of thin plate-like particles rather like minute particles of mica. In an undisturbed condition these particles lie parallel to each other. When wet, the particles slide readily over each other giving wet clay its slippery feel. As these clays dry out, suction forces develop between the plates producing the cohesion and the shrinking that are the unique characteristics of clayey soils.

During the weathering process the clays pass through different forms of hydrated alumino-silica. These can have quite different structures from kaolinite and this is one reason for the unusual properties of some tropical residual soils. One of the first clay minerals to form in hot humid climates is allophane. This has an amorphous gel-like structure and this structure gives it the ability to retain very large amounts of water. Two other minerals further down the line of evolution are halloysite and metahalloysite. With them, the laminar particle shape has emerged but the plates are curved into thin interlocking tubes, imparting a crumbly structure to the soil. Under different conditions montmorillonite can form. This clay mineral has plate like particles which are associated with very large volume changes in the soil between the wet and dry condition. These are the black cotton soils which occur in many tropical countries.

The form taken by the clay minerals is determined by the nature of the parent rock and by the local climatic and topographic conditions. The weathering of rocks to produce clays proceeds most rapidly in hot humid climates. Indeed, it is likely that the clays found in more temperate regions originated under tropical conditions and have evolved to a state in which the predominant clay minerals are kaolinitic. In the tropics these clay forming processes are still active in rocks, which are often deeply weathered, and it is the peculiar nature and engineering properties of these clays which is one of the promptings for this book.

Some soils are uniquely gravels, sands, silts or clays but most soils are a mixture of two or more of these particle size groups. Soil classification must take account of such mixtures. For this purpose a particle size analysis of the coarse fraction of the soil i.e. the gravel and sand retained on the 63 micron sieve, is supplemented by other tests to define the characteristics of the fine fraction i.e. the silt and clay. In a thorough examination, the particle size distribution of this fine fraction is determined by a sedimentation test, but this is a laborious and time consuming test involving, amongst other things, a determination of the specific gravity of the silt and clay. In normal survey work this test is replaced by the Liquid Limit and Plastic Limit tests. These tests were originally developed by Atterberg for defining the moisture content range over which agricultural soils could be tilled. Now no longer used by agriculturalists, they have been taken over by engineers as a useful and rapid way of defining the plasticity characteristics of the silt and clay fraction of soils.

TABLE 2.1
THE EXTENDED CASAGRANDE SOIL CLASSIFICATION SYSTEM

1		2	3	4	5
MAJOR DIVISIONS		DESCRIPTION AND FIELD IDENTIFICATION	SUB-GROUPS	GROUP SYMBOL	APPLICABLE CLASSIFICATION TESTS (CARRIED OUT ON DISTURBED SAMPLES)
COARSE-GRAINED SOILS	Boulders and cobbles	Soils consisting chiefly of boulders larger than 8 in. in diameter or cobbles between 8 in. and 3 in. identifiable by visual inspection	Boulder gravels	—	Particle-size analysis
	Gravel and gravelly soils	Soils with an appreciable fraction between the 3 in. and No. 7 B.S. sieves. Generally easily identifiable by visual inspection. A medium to high dry strength indicates that some clay is present. A negligible dry strength indicates the absence of clay	Well graded gravel-sand mixtures, little or no fines	G W	Particle-size analysis
			Well graded gravel-sands with small clay content	G C	Particle-size analysis, liquid and plastic limits on binder
			Uniform gravel with little or no fines	G U	Particle-size analysis
			Poorly graded gravel-sand mixtures, little or no fines	G P	Particle-size analysis
			Gravel - sand mixtures with excess of fines	G F	Particle-size analysis, liquid and plastic limits on binder if applicable
	Sands and sandy soils	Soils with an appreciable fraction between the No. 7 and the No. 200 B.S. sieve. Majority of the particles can be distinguished by eye. Feel gritty when rubbed between the fingers. A medium to high dry strength indicates that some clay is present. A negligible dry strength indicates absence of clay	Well graded sands and gravelly sands, little or no fines	S W	Particle-size analysis
			Well graded sands with small clay content	S C	Particle-size analysis, liquid and plastic limits on binder
			Uniform sands, with little or no fines	S U	Particle-size analysis
			Poorly graded sands, with little or no fines	S P	Particle-size analysis
			Sands with excess of fines	S F	Particle-size analysis, liquid and plastic limits on binder if applicable
FINE-GRAINED SOILS Containing little or no coarse-grained material	Fine-grained soils having low plasticity (silts)	Soils with an appreciable fraction passing the No. 200 B.S. sieve, and with liquid limits less than 35. Not gritty between the fingers. Cannot be readily rolled into threads when moist. Exhibit dilatancy	Silts (inorganic) rock flour, silty fine sands with slight plasticity	M L	Particle-size analysis, liquid and plastic limits if applicable
			Clayey silts (inorganic)	C L	Liquid and plastic limits
			Organic silts of low plasticity	O L	Liquid and plastic limits from natural conditions and after oven-drying
	Fine-grained soils having medium plasticity	Soils with liquid limits between 35 and 50. Can be readily rolled into threads when moist. Do not exhibit dilatancy. Show some shrinkage on drying	Silty clays (inorganic) and sandy clays	M I	Particle-size analysis, liquid and plastic limits if applicable
			Clays (inorganic) of medium plasticity	C I	Liquid and plastic limits
			Organic clays of medium plasticity	O I	Liquid and plastic limits from natural conditions and after oven-drying
	Fine-grained soils having high plasticity	Soils with liquid limits greater than 50. Can be readily rolled into threads when moist. Greasy to the touch. Show considerable shrinkage on drying. All highly compressible soils	Highly compressible micaceous or diatomaceous soils	M H	Particle-size analysis, liquid and plastic limits if applicable
			Clays (inorganic) of high plasticity	C H	Liquid and plastic limits
			Organic clays of high plasticity	O H	Liquid and plastic limits from natural conditions and after oven-drying
Fibrous organic soils with very high compressibility		Usually brown or black in colour. Very compressible. Easily identifiable visually	Peat and other highly organic swamp soils	Pt	Moisture content

6	7	8	9	10	11
APPLICABLE OBSERVATIONS AND TESTS RELATING TO THE MATERIAL IN PLACE (OR CARRIED OUT ON UNDISTURBED SAMPLES)	VALUE AS A ROAD FOUNDATION WHEN NOT SUBJECT TO FROST ACTION	POTENTIAL FROST ACTION	SHRINKAGE OR SWELLING PROPERTIES	DRAINAGE CHARACTERISTICS	MAXIMUM DRY DENSITY AT OPTIMUM COMPACTION (lb./cu. ft.) AND VOIDS RATIO, e *
Dry density and relative compaction Moisture content and voids ratio Cementation Durability of grains Stratification and drainage characteristics Ground-water conditions Large-scale loading tests California bearing ratio tests, shear tests and other strength tests	Good to excellent	None to very slight	Almost none	Good	—
	Excellent	None to very slight	Almost none	Excellent	>125 e $<0{\cdot}35$
	Excellent	Medium	Very slight	Practically impervious	>130 e $<0{\cdot}30$
	Good	None	Almost none	Excellent	>110 e $<0{\cdot}50$
	Good to excellent	None to very slight	Almost none	Excellent	>115 e $<0{\cdot}45$
	Good to excellent	Slight to medium	Almost none to slight	Fair to practically impervious	>120 e $<0{\cdot}40$
	Excellent to good	None to very slight	Almost none	Excellent	>120 e $<0{\cdot}40$
	Excellent to good	Medium	Very slight	Practically impervious	>125 e $<0{\cdot}35$
	Fair	None to very slight	Almost none	Excellent	>100 e $<0{\cdot}70$
	Fair to good	None to very slight	Almost none	Excellent	>100 e $<0{\cdot}70$
	Fair to good	Slight to high	Almost none to medium	Fair to practically impervious	>105 e $<0{\cdot}60$
Dry density and relative compaction Moisture content and voids ratio Stratification, fissures, etc. Drainage and ground-water conditions Consolidation tests Large-scale loading tests California bearing ratio tests, shear tests and other strength tests	Fair to poor	Medium to very high	Slight to medium	Fair to poor	>100 e $<0{\cdot}70$
	Fair to poor	Medium to high	Medium	Practically impervious	>100 e $<0{\cdot}70$
	Poor	Medium to high	Medium to high	Poor	>90 e $<0{\cdot}90$
	Fair to poor	Medium	Medium to high	Fair to poor	>100 e $<0{\cdot}70$
	Fair to poor	Slight	High	Fair to practically impervious	>95 e $<0{\cdot}80$
	Poor	Slight	High	Fair to practically impervious	>95 e $<0{\cdot}80$
	Poor	Medium to high	High	Poor	>100 e $<0{\cdot}70$
	Poor to very poor	Very slight	High	Practically impervious	>90 e $<0{\cdot}90$
	Very poor	Very slight	High	Practically impervious	>100 e $<0{\cdot}70$
and consolidation tests	Extremely poor	Slight	Very high	Fair to poor	—

* These unit weights apply only to soils with specific gravities ranging between 2·65 and 2·75

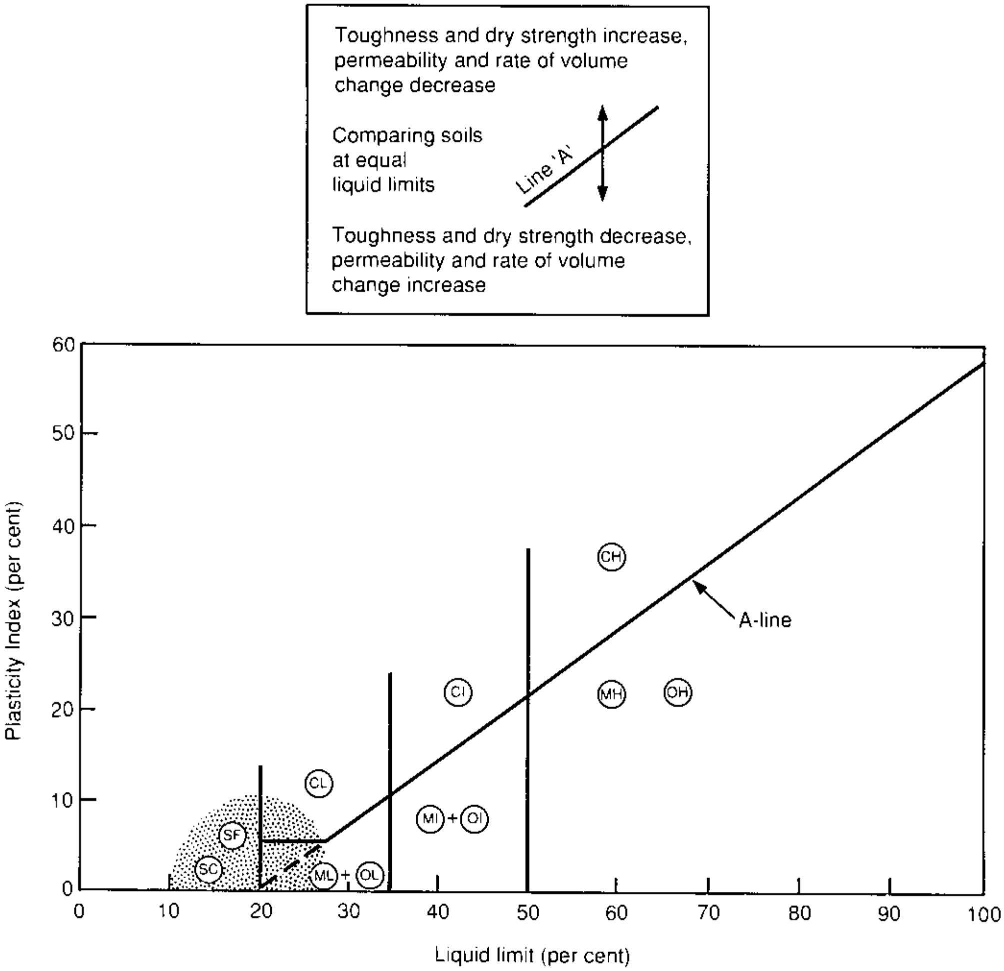


Fig.2.1 Plasticity chart used in Casagrande soil classification.

An engineering classification of soils based on these mechanical tests was first produced by Casagrande at the time when soil mechanics was emerging as an engineering science. Table 2.1 shows this classification which uses particle size analysis and the Liquid and Plastic Limits to classify soils into 21 groups, each with its own characteristic engineering properties. This classification is shown graphically in Fig. 2.1. The 'A' line in this diagram is particularly significant in separating soils of different strength, permeability and different propensity to change in volume with changing moisture content. The indications of the 'A' line are useful with transported soils but with some residual soils, notably those with an impermanent structure, these indications may be misleading.

In the USA, a simpler classification, developed by the American Association of State Highway and Transport Officials, is now in more general use. In this method, soils are divided into seven

TABLE 2.2
CLASSIFICATION OF SOILS AND SOIL-AGGREGATE MIXTURES (GRANULAR MATERIALS)

General Classification	Granular Materials (35% or less passing 0.075mm)						
Group Classification	A-1		A-3	A-2			
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7
Sieve Analysis. Percent passing:							
2.00mm (No. 10)	50 max.	...	...	...	...	...	...
0.425mm (No. 40)	30 max.	50 max.	51 min.	...	...	...	...
0.075mm (No. 200)	10 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.
Characteristics of Fraction passing 0.425mm (No. 40)							
Liquid limit	...		...	40 max.	41 min.	40max.	41 min.
Plasticity index	6 max		N.P.	10 max.	10 max.	11 min.	11 min.
Usual Types of Significant Constituent Materials	Stone Fragments Gravel and Sand		Fine Sand	Silty or Clayey Gravel and Sand			
General Rating as Subgrade	Excellent to Good						

major groups, A1 to A7 (Table 2.2) with further subdivision of groups A1, A2 and A7 (Table 2.3). In the United Kingdom, the Cassagrande Classification has been used to produce an extended system which retains the use of initials which correspond to the descriptive terms used in every day speech e.g. GW represents a well graded gravel, GC a clayey gravel, SC a clayey sand and so on (Table 2.4) (see also BS 5930 (1981)).

A classification of soils into some twenty groups is useful at the survey stage for indicating the road building strategies on a particular route and in establishing the form of the detailed testing necessary to evaluate the engineering properties of the soils. These properties, and the tests to evaluate them, are reviewed below. No attempt is made to describe the apparatus and the methods in detail. The intention here is to describe them only to the extent necessary to appreciate the purposes for which they are used and any idiosyncrasies which occur in their use. Many countries have National Standards covering test procedures for soils and other road making materials. Where available, such National Standards should be used. In this book, references are given to the test procedures defined in the appropriate British and American Standards.

It is important that the instructions in the appropriate standard be followed precisely to insure that the test results are accurate, giving a correct indication of the engineering properties and uses of the materials. These standards are under continuous revision to improve the accuracy and usefulness of the test results and new editions are produced from time to time incorporating new

TABLE 2.3
CLASSIFICATION OF SOILS AND SOIL-AGGREGATE MIXTURES (SILT-CLAY MATERIALS)

General Classification	Silt-Clay Materials (More than 35% passing 0.075 mm)			
Group Classification	A-4	A-5	A-6	A-7 A-7-5 A-7-6
Sieve Analysis. Percent passing:				
2.00mm (No. 10)	...	...	...	...
0.425mm (No. 40)	...	...	...	...
0.075mm (No. 200)	36 min.	36 min.	36 min.	36 min.
Characteristics of Fraction passing 0.425mm (No 40)				
Liquid limit	40 max.	41 min.	40 max.	41 min.
Plasticity index	10 max.	10 max.	11 min.	11 min.[a]
Usual Types of Significant Constituent Materials	Silty Soils		Clayey Soils	
General Rating as Subgrade	Fair to Poor			

a Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30

developments. Increasingly, these take into account the precautions needed in testing soils with the fragile structures which occur in tropical residual formations. The dates of the most recent revisions (up to 1991) are quoted and it is wise to check from time to time to learn about any subsequent revisions.

Many of the tests are in use in contractual situations to specify the properties required in the construction of particular works. This raises the issue of the confidence limits on the test results i.e. the repeatibility of test results obtained by a single operator and the reproducibility obtained by different operators on identical samples. With other road making materials, rocks, bitumens, asphalts and concrete for instance, the confidence limits on test results are usually known and can be employed in checking compliance with specifications. This is not so with soils. Their natural variability is one cause of scatter in the results and, in addition, some of the tests involve procedures that are not easy to reproduce with precision. Notes are given on how the results of these tests can be interpreted in contractual situations.

2.2 Survey and site investigations

BS 5930 (1981). Code of practice for site investigations.

American Association of State Highway Officials (AASHTO) Designation M145.82. Recommended practice for the classification of soils and soil aggregate mixtures for highway construction purposes.

TABLE 2.4
BRITISH SOIL CLASSIFICATION SYSTEM FOR ENGINEERING PURPOSES (FROM BS 5930)

Soil groups (see note 1)			Subgroups and laboratory identification				
GRAVEL and SAND may be qualified Sandy GRAVEL and Gravelly SAND, etc. where appropriate (see 41.3.2.2)			Group symbol (see notes 2 & 3)	Subgroup symbol (see note 2)	Fines (% less than 0.06 mm)	Liquid limit %	Name
COARSE SOILS less than 35% of the material is finer than 0.06 mm	GRAVELS More than 50% of coarse material is of gravel size (coarser than 2 mm)	Slightly silty or clayey GRAVEL	G GW	GW	0 to 5		Well graded GRAVEL
			GP	GPu GPg			Poorly graded/Uniform/Gap graded GRAVEL
		Silty GRAVEL	G-F G-M	GWM GPM	5 to 15		Well graded/Poorly graded silty GRAVEL
		Clayey GRAVEL	G-C	GWC GPC			Well graded/Poorly graded clayey GRAVEL
		Very silty GRAVEL	GF GM	GML, etc	15 to 35		Very silty GRAVEL; subdivide as for GC
		Very clayey GRAVEL	GC	GCL GCI GCH GCV GCE			Very clayey GRAVEL (clay of low, intermediate, high, very high, extremely high plasticity)
	SANDS More than 50% of coarse material is of sand size (finer than 2 mm)	Slightly silty or clayey SAND	S SW	SW	0 to 5		Well graded SAND
			SP	SPu SPg			Poorly graded/Uniform/Gap graded SAND
		Silty SAND	S-F S-M	SWM SPM	5 to 15		Well graded/Poorly graded silty SAND
		Clayey SAND	S-C	SWC SPC			Well graded/Poorly graded clayey SAND
		Very silty SAND	SF SM	SML, etc	15 to 35		Very silty SAND; subdivided as for SC
		Very clayey SAND	SC	SCL SCI SCH SCV SCE			Very clayey SAND (clay of low, intermediate, high, very high, extremely high plasticity)
FINE SOILS more than 35% of the material is finer than 0.06 mm	Gravelly or sandy SILTS and CLAYS 35% to 65% fines	Gravelly SILT	FG MG	MGL, etc			Gravelly SILT; subdivide as for CG
		Gravelly CLAY (see note 4)	CG	CLG CIG CHG CVG CEG		< 35 35 to 50 50 to 70 70 to 90 > 90	Gravelly CLAY of low plasticity of intermediate plasticity of high plasticity of very high plasticity of extremely high plasticity
		Sandy SILT (see note 4)	FS MS	MLS, etc			Sandy SILT; subdivide as for CG
		Sandy CLAY	CS	CLS, etc			Sandy CLAY; subdivide as for CG
	SILTS and CLAYS 65% to 100% fines	SILT (M-SOIL)	F M	ML, etc			SILT; subdivide as for C
		CLAY (see notes 5 & 6)	C	CL CI CH CV CE		< 35 35 to 50 50 to 70 70 to 90 > 90	CLAY of low plasticity of intermediate plasticity of high plasticity of very high plasticity of extremely high plasticity
ORGANIC SOILS		Descriptive letter 'O' suffixed to any group or sub-group symbol.	Organic matter suspected to be a significant constituent. Example MHO: Organic SILT of high plasticity.				
PEAT		PT	Peat soils consist predominantly of plant remains which may be fibrous or amorphous.				

NOTE 1. The name of the soil group should always be given when describing soils, supplemented, if required, by the group symbol, although for some additional applications (e.g. longitudinal sections) it may be convenient to use the group symbol alone.

NOTE 2. The group symbol or sub-group symbol should be placed in brackets if laboratory methods have not been used for identification, e.g. (GC).

NOTE 3. The designation FINE SOIL or FINES, F, may be used in place of SILT, M, or CLAY, C, when it is not possible or not required to distinguish between them.

NOTE 4. GRAVELLY if more than 50% of coarse material is of gravel size. SANDY if more than 50% of coarse material is of sand size.

NOTE 5. SILT (M-SOIL), M, is material plotting below the A-line, and has a restricted plastic range in relation to its liquid limit, and relatively low cohesion. Fine soils of this type include clean silt-sized materials and rock flour, micaceous and diatomaceous soils, pumice, and volcanic soils, and soils containing halloysite. The alternative term 'M-soil' avoids confusion with materials of predominantly silt size, which form only a part of the group.

Organic soils also usually plot below the A-line on the plasticity chart, when they are designated ORGANIC SILT, MO.

NOTE 6. CLAY, C, is material plotting above the A-line, and is fully plastic in relation to its liquid limit.

The American Standard applies particularly to surveys and site investigations for new roads. The British Standard covers site investigations for all civil engineering structures. Its application to highway engineering lies in the presentation of the soil classification system referred to above, in instructions given for soil sampling, and for noting site details of engineering significance e.g. prevailing moisture conditions, depth of water table, etc. To these latter might usefully be added evidence on the stability of natural slopes in the soils encountered along the route and any evidence of their resistance to erosion on exposed soil faces and in natural water channels.

2.3 Soil testing

The relevant British and American National Standards covering the apparatus and methods of testing soils are:-

BS 1377 (1990). Methods of test for soils for civil engineering purposes. This recent revision is divided into nine separate parts. In the descriptions of testing procedures below, references are given to the appropriate parts.

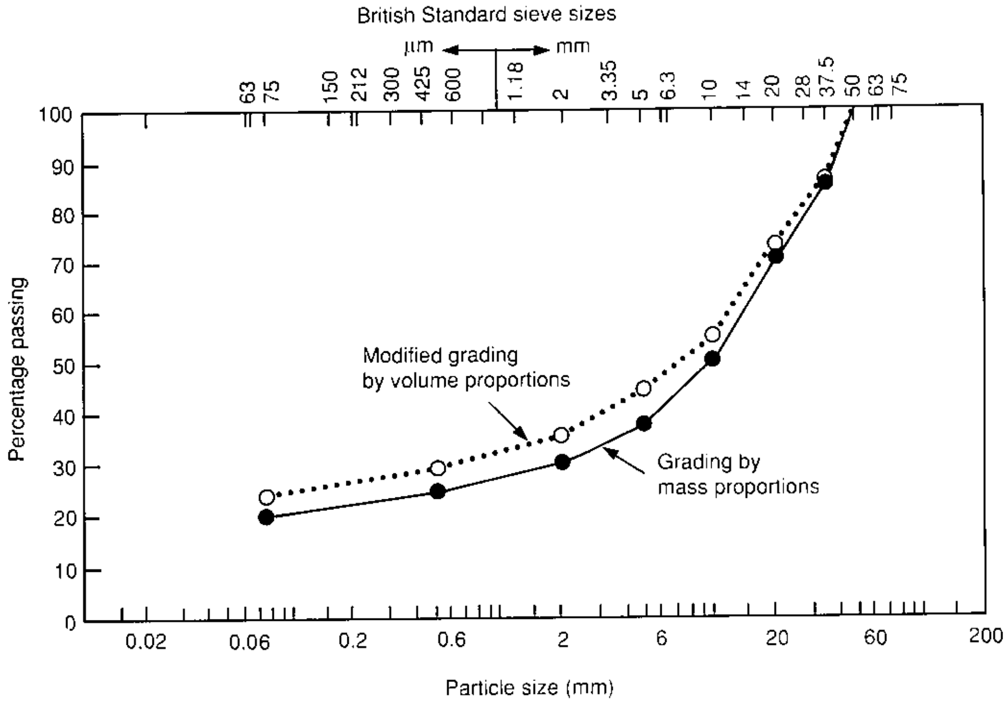
The American specifications, issued by the American Society for Testing Materials (ASTM), are mostly contained in the Annual of the ASTM, Volumes 04.02, 04.03 and 04.08. Some specifications specific to highway engineering are issued by the American Association of State Highway and Transport Officials (AASHTO) and again, appropriate references are given.

2.3.1 *Particle Size Analysis (coarse fraction)* BS 1377, Part 1; ASTM D422.63, and AASHTO T11 and T27.

The same sieve sizes are used, namely 75, 63, 50, 37.5, 28, 20, 14, 10, 3.35, 2, 1.18 millimetres and 600, 425, 300, 212, 150, 63 micrometres. Square-holed metal sieves are used for the larger sieves and woven wire sieves for the smaller sizes. Originally shaken by hand, the sieves are now almost invariably agitated in mechanical shakers. Wet sieving i.e. washing the soil through successive sieves with water, is preferred in the British Standard because it gives more reproducible results and a more accurate representation of the engineering characteristics of the soil. This is particularly important with some tropical residual soils because the fines tend to adhere to the larger particles. Dry sieving should be used only if it has been shown that the same results are obtained as with wet sieving. This effectively excludes many clayey soils.

The purpose of particle size analysis is to determine the proportional volume occupied by the particles of different sizes. Since the proportions are determined by weighing the dried material retained between each sieve, this involves the assumption that the specific gravity of the particles is constant over the range of sizes. With most soils this is a valid assumption, but with some soils, notably laterites, there can be large differences between the specific gravities of the coarse and fine fractions and a correction to the grading curves is necessary on this account. This is illustrated in Fig. 2.2.

Test sieves become worn by use. They should be regularly inspected and replaced as soon as signs of wear are apparent.



Assuming constant specific gravity (S.G.)

Sieve size (mm)	*Cumulative mass percentage passing each sieve*	*Therefore mass retained on each sieve in a 100g sample*	*Therefore mass retained on each sieve reproportioned for variation in S G.*	*Equivalent mass retained on each sieve for 100g sample*	*Therefore cumulative volume percentage passing each sieve*
50	100	0	0	0	100
31.5	85	15	12	14	86
20	70	15	12	14	72
10	50	20	15	17	55
5	38	12	9	10	45
2	30	8	8	9	36
0.5	25	5	5	6	30
0.075	20	5	5	6	24
		20	20	23	1
		100g	86g	99g	

But a visual inspection and separate S.G. determinations reveal that
S.G. (fraction > 5mm) = 3.5
S.G. (fraction < 5mm) = 2.7
Therefoe mass retained on each sieve larger than 5mm should be factored by 2.7:3.5 or 0.77

Fig.2.2 *Modification to particle size distribution required with lateritic gravels containing nodules of high specific gravity (Charman, 1988, CIRIA Special Publication 47).*

Sieve analysis is used during survey and design. It may also be used in control testing during construction, for example, with gravels to be used in road surfacings and bases. The standard specifications do not quote confidence limits for the accuracy of sieving tests but it may be assumed that provided the recommended test procedures are strictly followed (including the sampling procedure which is very important), the test gives results sufficiently reproducible for contractual purposes.

2.3.2 *Sedimentation Test* BS 1377, Part 2.

The test is undertaken whenever detailed information is required on the particle size distribution of the silt-clay fraction of a soil. It is an important research test but rarely finds use in routine civil engineering works. It involves complicated and tedious testing procedures and is included here so that readers can recognise it should they encounter it in detailed site investigations.

The analysis uses Stoke's Law which states that the rate of settlement of solid particles in a fluid is proportional to their size. The silt-clay fraction of the soil must first be dispersed so that the particles of different size can settle without interference from other particles. For this purpose a dispersing agent is used. Sodium hexametaphosphate has been found suitable for use with most tropical soils and, occasionally, it may be necessary to use a higher proportion than normal to ensure adequate dispersal. The soil is dispersed in distilled water in a tall glass cylinder and shaken to make sure that the soil particles are evenly distributed through the suspension. Thereafter, their rate of settlement is observed. In the simplest method this settlement is measured by recording the rate of fall of a hydrometer as the density of the soil-water mixture at the top of the cylinder decreases by settlement of the soil particles towards the bottom. In a more elaborate and accurate method, a pipette is used to draw samples from a particular depth at intervals over a period up to 7.5 hours.

This test may sometimes be used in design testing and also in detailed failure investigations. It is quite unsuitable for use as a control and compliance test during construction.

2.3.3 *Liquid and Plastic Limits* BS 1377, Part 2; ASTM D4318.

The British and American methods are similar and are illustrated in Figures 2.3 and 2.4. These tests are done on the soil fraction passing the 425 micrometre sieve. The purpose of the Liquid Limit Test is to define the moisture content at which the wet soil fines pass from a plastic to a liquid condition. The soil fines are wetted to a fairly high moisture content and a pat of the soil is placed in the cup of the testing machine. A groove is then cut in the pat from front to back with a special grooving tool. The cup is then raised by a rotating cam and allowed to fall sharply on to a hard rubber base as the handle of the apparatus is turned. The number of impacts required to close the groove is observed. The test is then repeated with the soil at successively higher moisture contents until it is possible to estimate the moisture content at which the groove would be closed by 25 blows. This moisture content is recorded as the Liquid Limit.

A one-point method is used by skilled operators, who develop the faculty of judging the moisture content at which the soil fines are close to the liquid limit. In this method, use is made of a table of conversion factors to indicate the Liquid Limit from single tests in which the groove in the pat has closed after between 15 and 35 blows. This method should be used only on soils for which experience has shown that the conversion factors are accurate, and only when speed of testing is important and some loss of accuracy is acceptable.

The same sample of soil is then used for the Plastic Limit Test. After further drying, the soil is rolled out on a glass plate under the palm of the hand to form a thin cylinder approximately 6 millimetres in diameter. Rolling is continued to compress the thread of soil to 3 millimetres diameter. The test is continued as the soil dries out and, as the moisture content is reduced, a stage

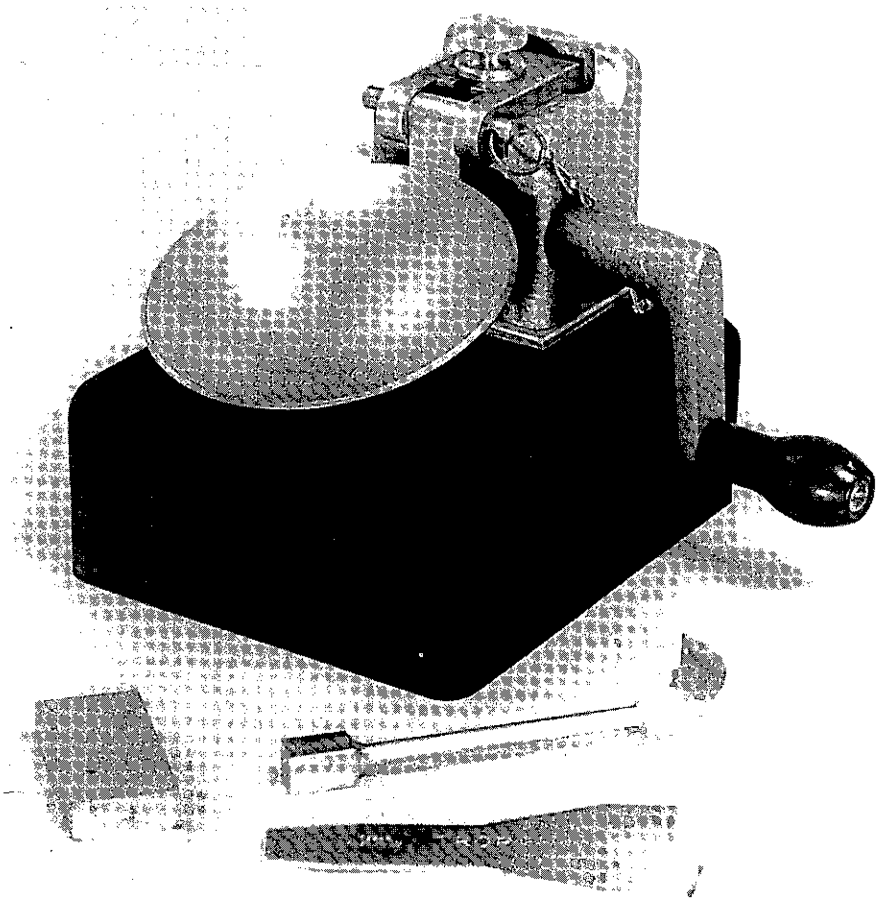


Fig.2.3 The Liquid Limit Test.

is reached when the thread crumbles into separate particles. The moisture content at this stage is recorded as the Plastic Limit.

The value of the Liquid Limit indicates the capacity of the soil fines to retain moisture, and the difference between the Liquid and Plastic Limit, the Plasticity Index (PI), serves as a proxy for the clay content of the soil fines.

In an attempt to simplify the Liquid Limit Test and to make it more reproducible, an alternative method of test is preferred in the British Standard. This involves using the penetrometer apparatus employed in testing refined bitumens (Figs. 5.5 and 5.6) but replacing the test needle with a steel cone 35 millimetres long and with an angle of 20°. Again, tests are done over a range of moisture contents, the Liquid Limit being interpolated as the moisture content at which the cone penetrates 20 millimetres into the soil in 5 seconds under the standard load of 100 grammes. There is, as yet, no more reliable alternative testing procedure for the Plastic Limit Test.

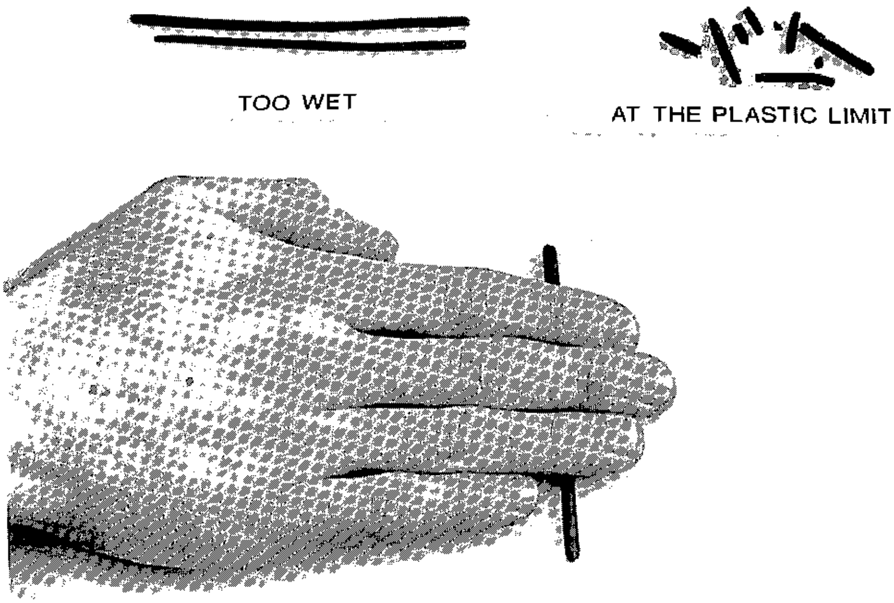


Fig.2.4 The Plastic Limit Test.

The Plastic and Liquid Limit Tests have the virtue of being simple and quite rapid. With a skilled operator they can give fairly consistent results but there can be quite large differences in the results obtained by different operators, particularly with the Plastic Limit Test. Thus, whilst these tests are useful in survey and in evaluating the likely potential of gravel deposits, it is unwise to rely on their accuracy in contract specifications. Unfortunately, for lack of something better, they are frequently used in specifying gravels and crushed stone bases. Here, the best advice is to take particular care in sampling, making sure that the only valid tests for contractual purposes are done on samples that the engineer and contractor agree are representative.

As with particle size analysis, these tests can give anomalous results with tropical soils which possess a structure that is altered by heat and by working. Prominent amongst such soils are clays deriving from the weathering of volcanic ash, and examples of these effects are shown in Fig. 2.5 and in Table 2.5. For classification purposes it has been suggested that such soils should be heavily worked until their structure is completely broken down before they are tested. But this would defeat the engineering purpose of the classification, since it is vital to be aware of the presence of this structure and how it can be employed, if possible to advantage, during construction.

TABLE 2.5
EFFECTS OF DRYING ON PLASTICITY CHARACTERISTICS OF SOME EXAMPLES OF TROPICAL RESIDUAL SOILS

Soil source and reported pedological type	Atterberg Limits					
	Natural		*Air dried*		*Oven dried*	
	LL	PL	LL	PL	LL	PL
Costa Rica Laterite	81	29	-	-	56	19
Dominica Allophane	101	69	56	43	-	-
Hawaii						
Humic Latosol	164	162	93	89	-	-
Hydrol Latosol	206	192	61	NP	-	-
Java						
Andosol	184	106	-	-	80	74
Kenya (Sasumua)						
Red Clay	101	70	77	61	65	47
Malaysia						
Weathered Basalt	115	50	-	-	69	49
New Guinea Andosol	145	75	-	-	NP	NP
Vanuatu						
Pentecost Weathered Volcanic Ash	261	184	192	121	NP	NP

NP = Non-plastic

In many tropical countries there is a need for a diagnostic test to indicate the presence of montmorillonitic clays. The Plasticity Index may be used for this purpose. Fig. 2.6 shows the relationship between the Plasticity Index and the clay content of a range of soils. The consistency of the relationship with most British soils is an indication of the maturity of clay soils in northern latitudes, and the scatter with the tropical soils illustrates the effects of the special clay minerals present in some of these soils. Those above the line are volcanic clays containing halloysitic clays whilst those below the line, including the British outlier, contain montmorillonite. Shrinkage tests, described below, can also be a useful indicator of the presence of montmorillonite clay.

2.3.4 *Linear Shrinkage* BS 1377, Part 4; ASTM D42.7.

The British and American test procedures are similar. Volumetric shrinkage test procedures are also described in the Standards. This test has been selected for special mention because of its simplicity and because it gives a useful indication of clay content of soil fines combined with their susceptibility to volume change with changes in moisture content.

In this test a small metal trough is used, as illustrated in Fig. 2.7. This metal trough is filled level with the top of the trough with soil fines at a moisture content near the Liquid Limit. The specimen is then carefully dried, first in air, then at 60-65°C until shrinkage is substantially complete, and

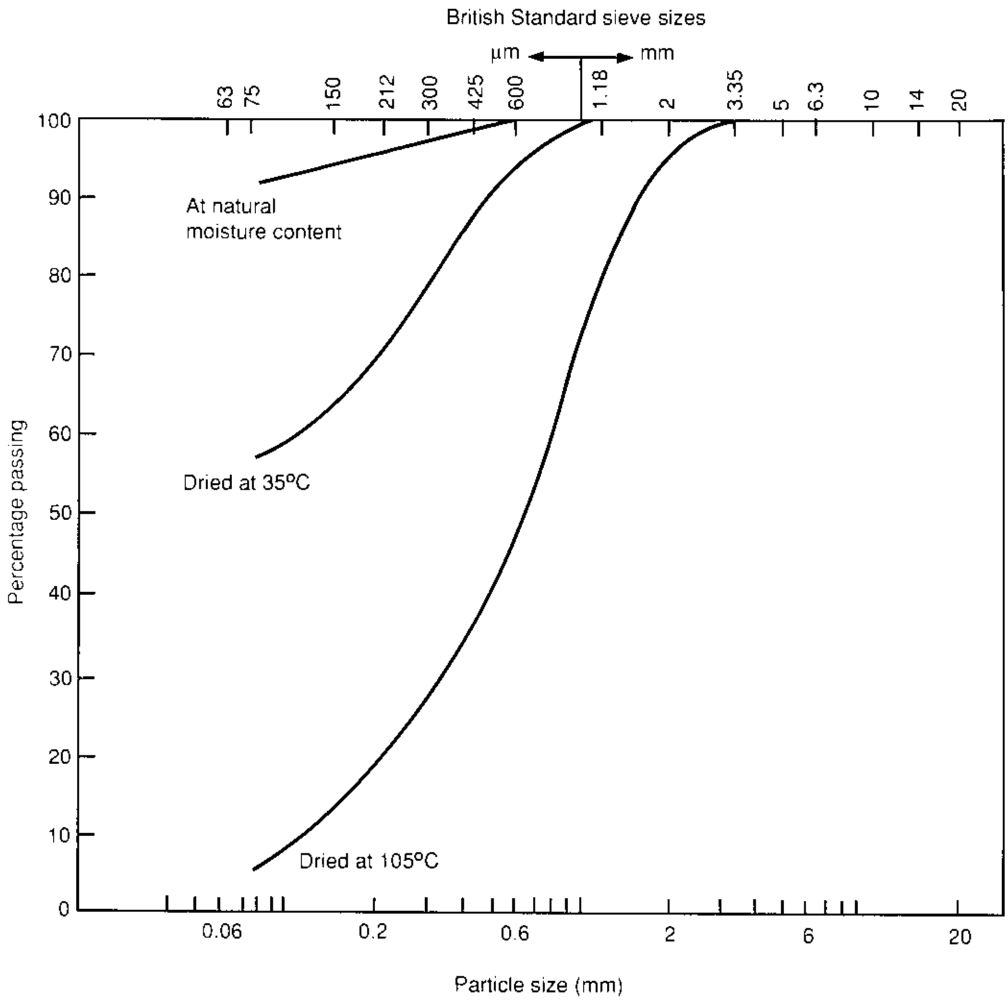
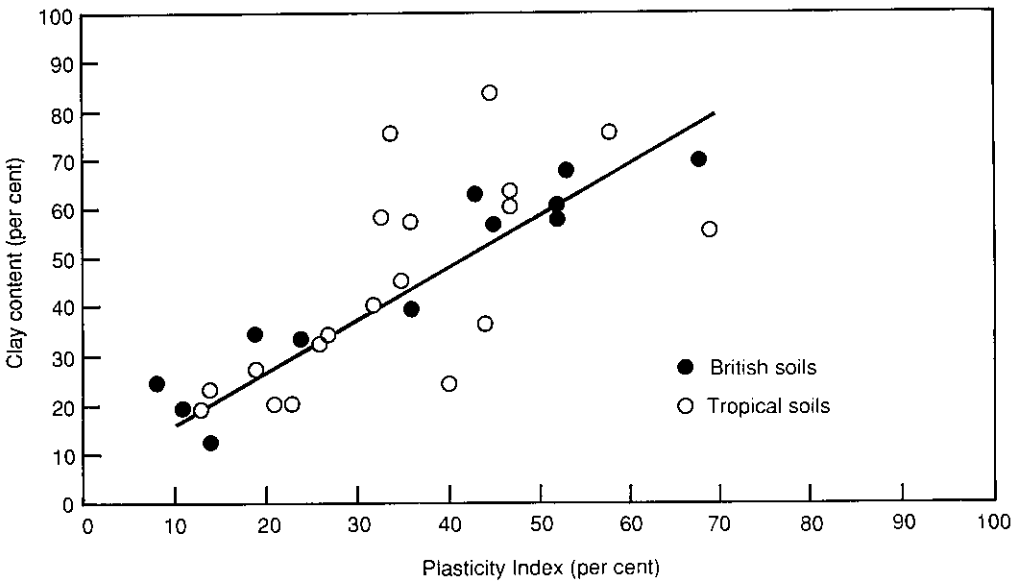


Fig.2.5 *The effect of drying temperature on the size distribution of a weathered volcanic ash (from Pentecost, Vanuatu).*

finally at 105-110°C. After the specimen has cooled, its shrunken length is measured and the Linear Shrinkage is defined as the percentage reduction in length of the specimen.

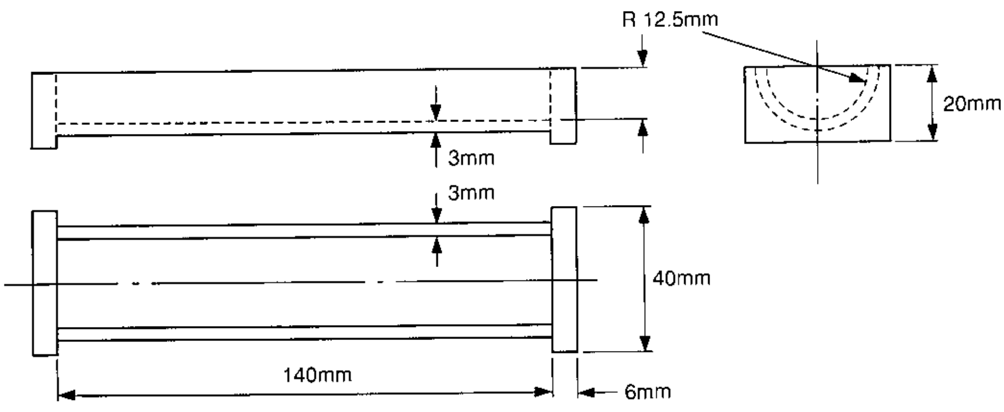
No confidence limits are quoted for the accuracy of this test but repeatability and reproducibility are likely to be somewhat better than the Plastic Limit Test. Its chief use is in survey and design testing but it does occasionally occur in specifications e.g. to characterise the fines in gravels used for road bases and surfacings.

Clayey soils expand and contract with changing moisture content. A typical relationship between volume and moisture content for such soils is shown in Fig. 2.8. As the soil dries from a wet condition, the relationship is linear, following the zero air voids line until the larger soil particles



British soils + Tropical soils

Fig.2.6 Relationship between clay content and Plasticity Index for various soils.



Material: non-ferrous

Fig.2.7 Mould for the Linear Shrinkage Test.

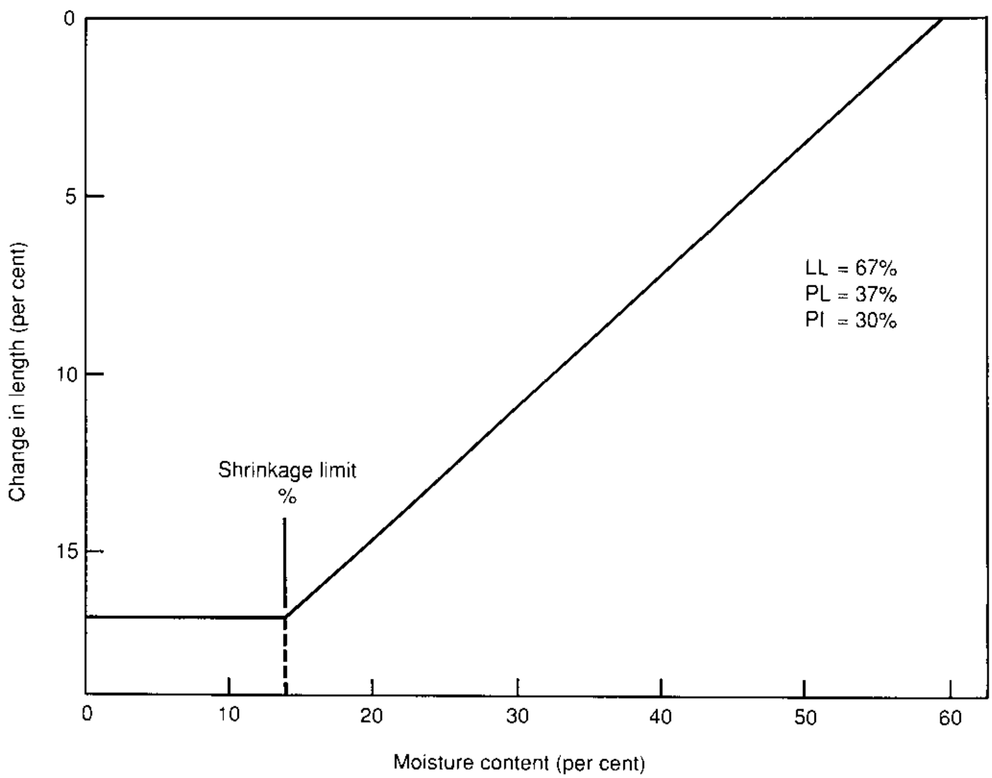


Fig.2.8 *Relationship between shrinkage and moisture content for a clay soil.*

come into contact. Thereafter, air enters the soil to replace the water which is removed and the shrinkage of the soil ceases. In this dry condition, large suction forces are developed which impart the cohesion that is characteristic of clayey soils. The intercept of the zero air voids line and the horizontal line representing no further volume change, indicates the moisture content known as the Shrinkage Limit. The American and British Standards also describe volume shrinkage tests on cylindrical soil samples involving displacement by mercury to measure the change in volume on drying. An alternative optical method, described by Croney *et al* (1958), may be preferred because it does not involve any external loading on the specimen.

Montmorillonitic clays have particularly low shrinkage limits and it is this which is responsible for the high volume changes that can occur with these soils between wet and dry seasons.

2.3.5 *Specific Gravity* BS 1377, Part 2; ASTM C127 and C128.

This test is used in survey work and in testing for design. The results may also be needed during construction to obtain a more complete picture of the state of compaction of soil than is given by relative density measurements. There are differences between the British and American testing methods. Many soils contain particles of different specific gravities and therefore it is necessary to make separate determinations on the coarse and fine fractions of the soil. The repeatibility of the test varies according to the uniformity of the soils and it may be desirable to undertake a sufficient number of tests to indicate the mean value and range of specific gravities for a particular soil.

2.3.6 *Field Density* BS 1377, Part 9; ASTM D1556-90, D2937-83, D3017-88.

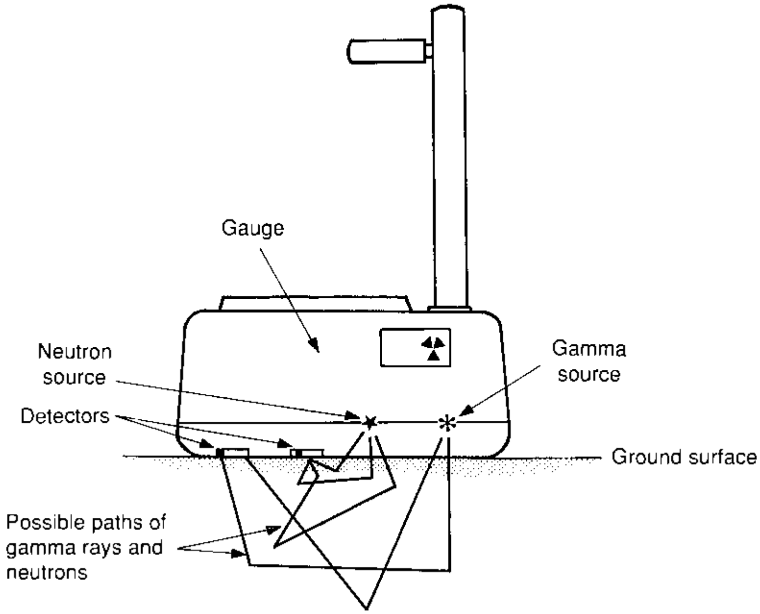
We now move into a series of tests which are used to define specific engineering properties of soils and it will be useful to outline their purposes. Field density measurements are made during survey as part of soil classification but during construction they have another purpose. Soil compaction is an important component of construction. It reduces to a minimum any future settlement of the soil under the road and it increases the load carrying capacity to the maximum possible level. The latter is particularly important in the top metre of soil immediately under the pavement.

Three methods of measuring the field density of soils are in common use and the testing procedures for all three are defined in the National Standards. The simplest is the core-cutter method (or drive-cylinder method). This method can be used only on cohesive soils free from coarse-grained material. It involves driving a hollow metal cylinder, which has a cutting edge, into the soil to remove an undisturbed sample on which dry density and moisture content determinations can be made.

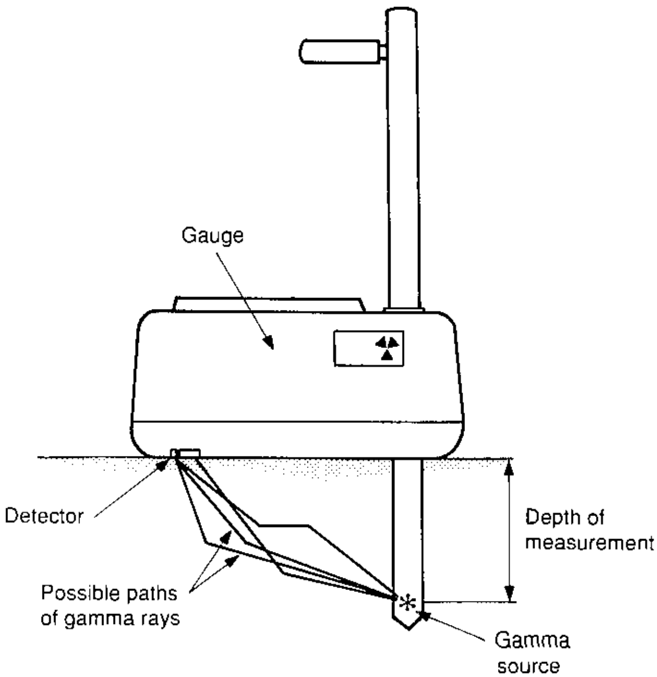
Most widely used is the sand replacement method in which a cylindrical hole is carefully excavated in the soil to approximately standard dimensions. The dry weight and moisture content of the soil removed from the hole are determined and the hole then filled with a standard single-sized sand using apparatus by which the volume of sand filling the hole can be estimated. This is rather a laborious process and the search has been going on for many years to develop an accurate and instantaneous method using nuclear radiation. Both National Standards now contain specifications for such a method, including recommendations on safety precautions, and the apparatus and materials are commercially available. Some of these operate by direct transmission and others by back-scatter. These two modes are illustrated in Fig. 2.9. Direct transmission is preferred because of its deeper zone of influence.

The apparatus provides an indirect way of measuring both the density and the moisture content of the soil, using gamma rays to indicate the bulk density and neutron moderation to indicate moisture density. The manufacturers of the apparatus supply calibration data from which dry densities and moisture contents of the soil can be derived. The method has limitations, the results being affected by the chemical composition of the soil. To date the method finds its main use in control testing rather than in testing for compliance to a specification but it can be used for this latter purpose provided the results are carefully calibrated with direct measurements of density and moisture content of the soils encountered on the site.

The Standards do not give quantitative information on the accuracy of the test methods. This would be impracticable since accuracy varies with the type of soil, being at its best with fine-textured uniform soils and worst with well graded soils containing coarse particles. A large number of tests is necessary with these latter soils (and with crushed rock used in road bases) to



(a) Back-scatter method



(b) Direct method

Fig.2.9 Two types of nuclear densimeters.

obtain an accurate indication of their state of compaction and this is one reason why, in the United Kingdom, a method specification rather than an end product specification is frequently used for securing adequate compaction of soils and other pavement materials (see Chapter 6).

2.3.7 *Laboratory Compaction* BS 1377, Part 4; ASTM D 698 and D1557.

These tests are undertaken to determine the combined influences of compactive effort and moisture content on the dry density of soils. Typically, such tests produce a curve of inverted U shape indicating an optimum moisture content for maximum compaction with the particular compactive effort employed (Fig. 2.10). On the wet side of this optimum, the presence of an increasing volume of water physically prevents further compaction of the soil. On the dry side of the optimum, the diminishing presence of water reduces its lubricating affect. With some soils the state of compaction increases again as the moisture content moves towards zero, presumably because there is insufficient water present to mobilise the suction forces from which clayey soils derive their cohesion. This phenomenon is illustrated in some of the compaction curves in Fig. 2.10 and it finds useful application in dry tropical climates where water is not readily available to assist in field compaction (see Chapter 11).

The compaction tests were first evolved in the USA. They are used all over the world, first to evaluate the compaction characteristics of soils and then to specify the state of compaction to be achieved in soil embankments and foundations. For this latter use, the term Relative Density is used i.e. the minimum acceptable state of compaction is specified as a percentage, normally between 90 and 100 per cent, of the maximum dry density achieved in a defined laboratory compaction test.

These laboratory tests consist of measuring the dry density of soil after compaction in a cylindrical mould by a specified number of blows from a hammer of standard weight and dimensions falling from a standard height, the whole process being repeated for a range of moisture contents. As first developed, this operation was done manually. For some years now, it has been mechanised so that the movements of the hammer are more closely controlled, and the mould is rotated horizontally so that the compactive effort is evenly distributed over the surface of the soil.

Two compactive efforts are in use. In American terminology they are known as Proctor and Modified AASHTO and in the British Standard they are known as Light and Heavy. In the former, a hammer of 2.5 kilograms falls for 300 millimetres, and in the latter, a 4.5 kilogram hammer falls for 450 millimetres. The number of hammer blows is prescribed for each test.

These laboratory compaction procedures provide only an approximate simulation of the stress regimes under different types of field compaction equipment. Several considerations flow from this. A wide range of field compaction equipment has been developed including smooth steel-wheeled, rubber-tyred, sheepsfoot, grid and impact rollers. Many of these are fitted with vibration equipment, the amplitude and frequency of which can be varied. It is always best to select the type of equipment most suitable for particular soils, carrying out field trials, if necessary, to modify the indications of the laboratory compaction tests for the specifications. More information is given on this in Chapter 11.

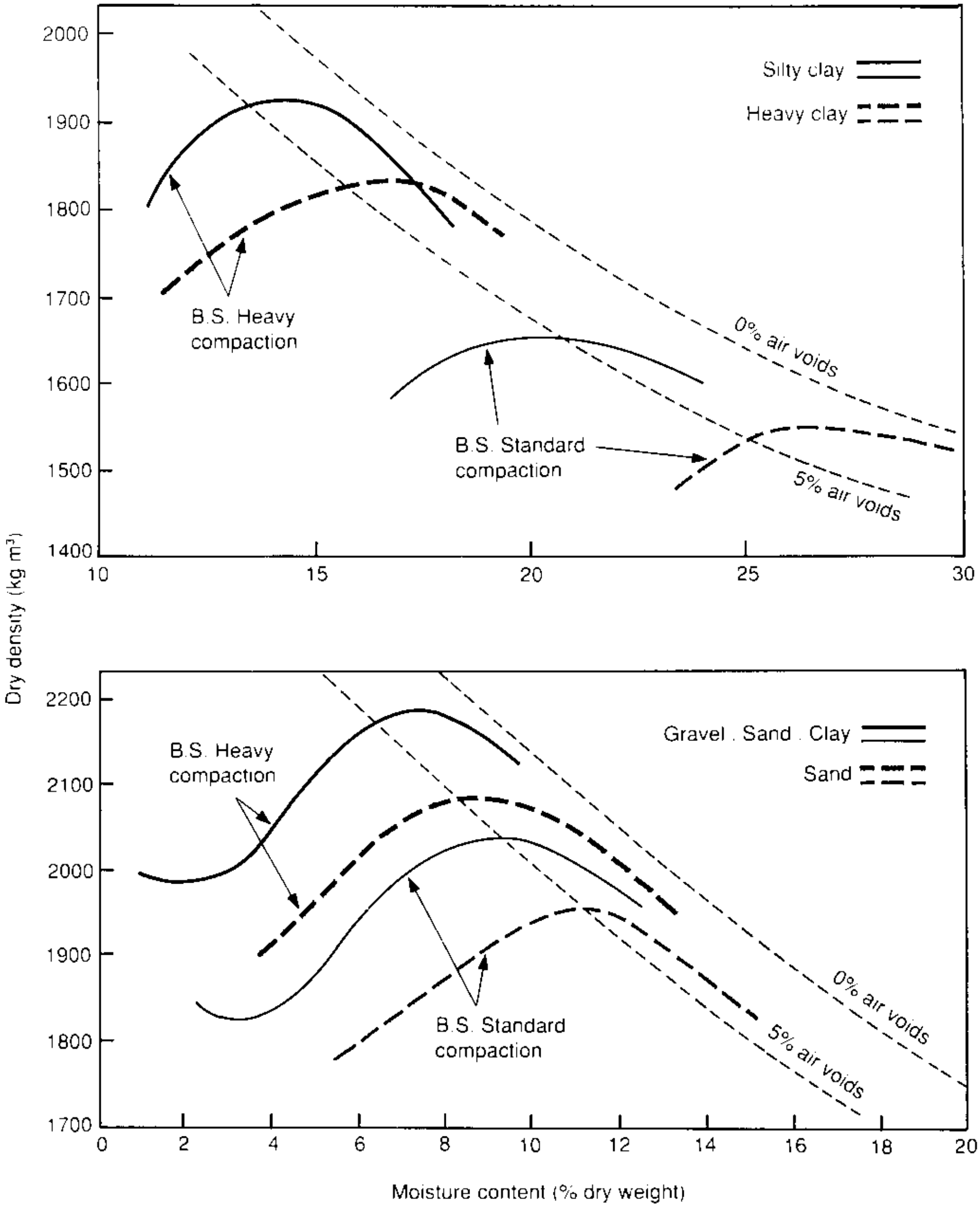


Fig.2.10 The effect of compactive effort on the relationship between dry density and moisture content for different soil types.

The laboratory tests are not good indicators of field behaviour with soils of low plasticity for which high frequency vibratory compaction is particularly suitable. With this in mind, it may be more appropriate to use vibratory compaction rather than hammer compaction when evaluating these soils in the laboratory. BS 1377 includes a form of test using vibratory compaction for use with sandy soils of low plasticity.

Many tropical residual soils have a structure which is at least partially destroyed by working. For example, the coarse particles in concretionary gravels can vary in strength between deposits, and soils containing halloysitic clay minerals can be broken down and weakened by heavy compaction. Compaction test procedures are defined in BS 1377, Part 4 for use with soils having particles susceptible to crushing. With such soils, an indication of their vulnerability can be obtained by undertaking particle size analysis on samples before and after they go through the compaction test procedure. Field compaction trials are often likely to be needed with such soils. One of the advantages of the pedological approach to soil classification, described later in this chapter, is that it provides a framework within which experience can be accumulated on the most suitable methods of compacting the different soils occurring in a particular region.

The Standards give no quantitative information on the confidence limits to be used in applying the results of laboratory compaction tests in contractual situations. Nevertheless the testing methods are not complicated and, provided their limitations are taken into account and the test procedures are rigorously followed, the test results can be used with full confidence.

2.3.8 *California Bearing Ratio* BS 1377, Parts 4 and 9; ASTM D1883.

In this test, laboratory compacted specimens are evaluated to give a numerical value related to the support the soil will give for a pavement i.e. in pavement design. The test is also used to evaluate the strength of gravel sub-bases and bases. A cylindrical metal plunger of 50 millimetres diameter is made to penetrate the specimen at a rate of one millimetre per minute and readings of the force generated between the plunger and the specimen are taken at intervals of penetration of 0.25 millimetres up to a total penetration of not more than 7.5 millimetres. The test apparatus is illustrated in Fig. 2.11.

The CBR value is reported as a percentage of a standard value which is intended to represent the value that would be obtained with compacted crushed stone. (This reflects the origin of the test as a means of evaluating bases for use under bituminous surfacings). Typical test results are illustrated in Fig. 2.12. The American and British test apparatus are similar. As originally conceived, the test was undertaken on specimens that had been immersed in water for four days. The American procedure continues to emphasise soaking as an essential feature. In the British procedure the intention is to explore the effects of the state of compaction and moisture content on the CBR value in order to provide data from which an appropriate value can be derived for use in pavement design. A test procedure involving soaking for four days is included in BS 1377 for those who wish to use it.

CBR tests are undertaken on laboratory samples of soil and other available road making materials during survey and design. With the soil subgrade these tests are used in designing the pavement and in indicating the level of compaction to be specified for the subgrade. These aspects are discussed more thoroughly in subsequent chapters on pavement design and compaction. For sub-bases and bases it is usual to specify minimum CBR values, 30 per cent and 50-100 per cent being typical values required. The Standards provide no guidance on the confidence limits to be placed on test results when used for contractual purposes. Experience suggests that the test is of poor reproducibility, particularly with granular soils. Where replicate testing is to be undertaken, it is essential that the test samples used by different operators be obtained by careful division of one larger sample which all the parties concerned agree is representative.

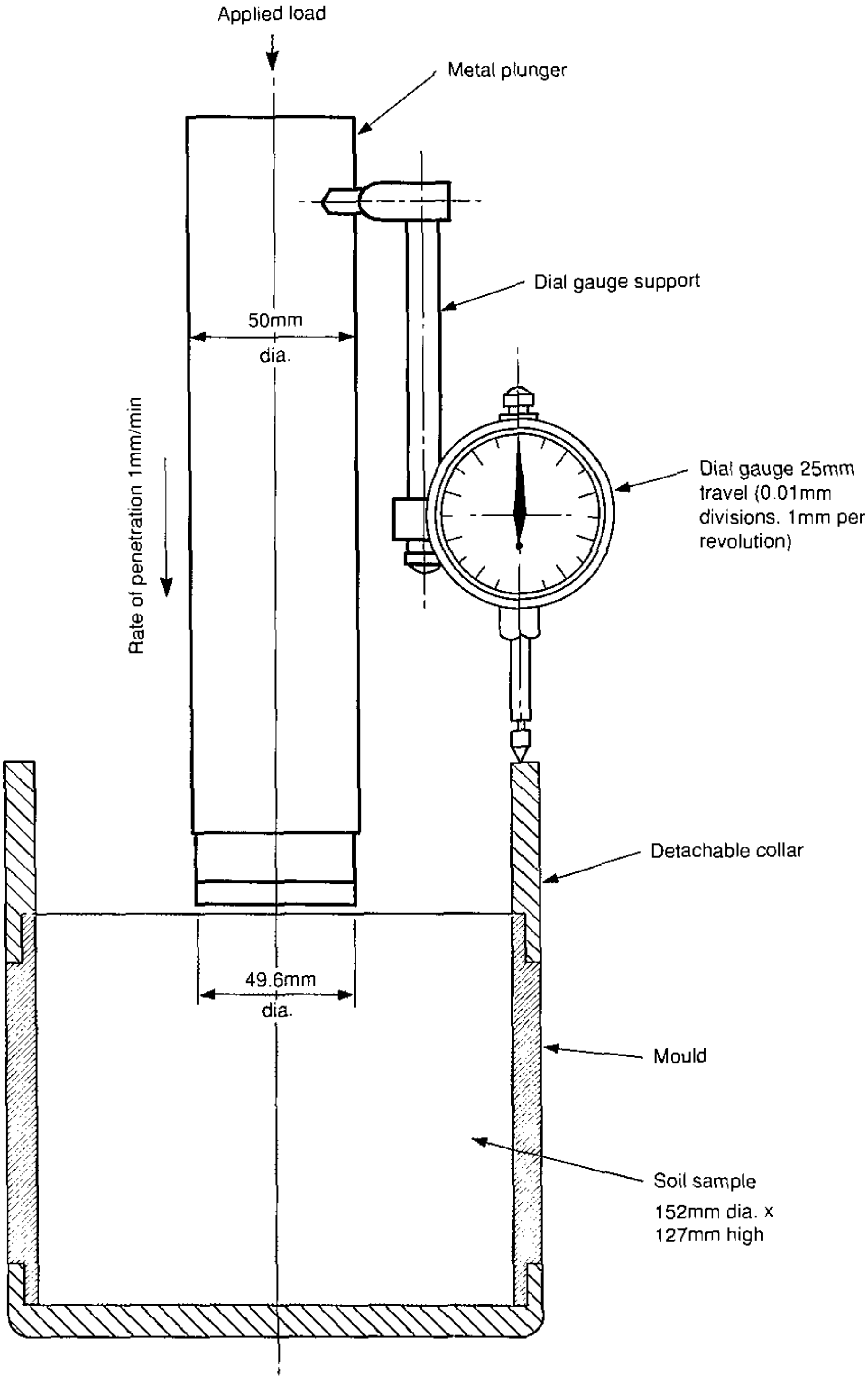


Fig.2.11 Apparatus for the California Bearing Ratio Test.

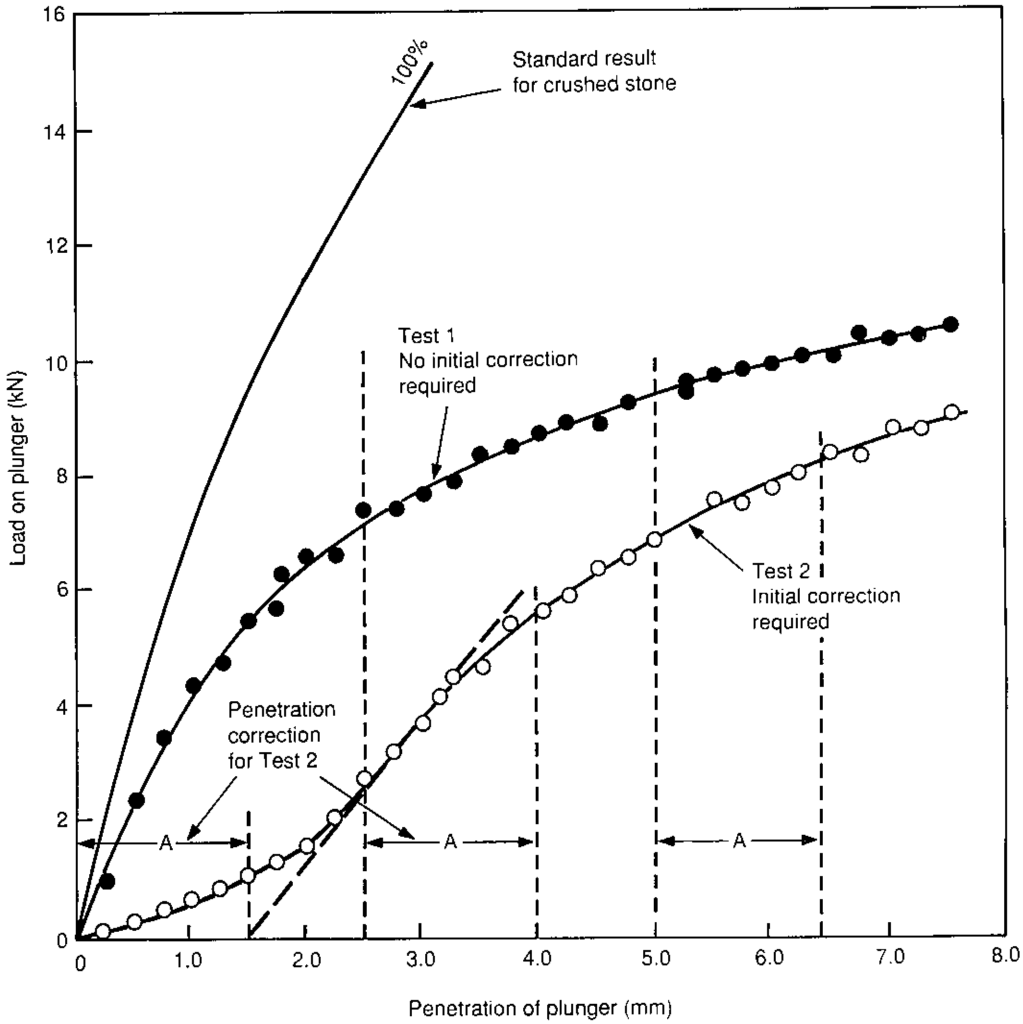


Fig.2.12 *Typical results from the CBR Test.*

The testing of soaked samples suggests that the engineer has no confidence in his ability to design and build a road structure within which surface water will not accumulate. But even on this pessimistic assumption, soaking for four days can give an unrealistic indication of the bearing capacity of a soil. With clayey soils of low permeability, the saturating water may not penetrate very far into the sample and soils of low or zero plasticity are likely to lose water rapidly on removal from immersion. With subgrade soils and with bases and sub-bases, it is more useful to derive CBR values for design purposes from tests on samples that are at the densities and the estimated critical moisture contents likely to occur in the road.

Equipment is available to carry out in situ CBR tests in the field on exposed subgrades, sub-bases and bases. Procedures are described in BS 1377, Part 9 and typical apparatus is illustrated in

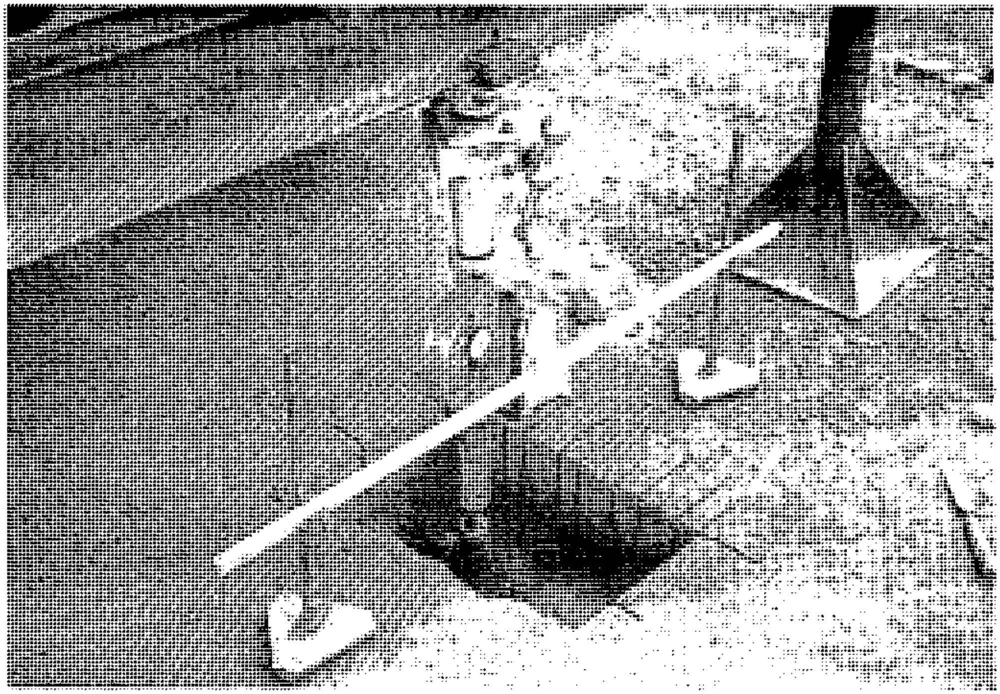


Fig.2.13 In situ CBR apparatus.

Fig. 2.13. Such testing can be useful in investigating pavement failures and also in examining existing roads in good condition. Accompanied by measurements of field densities and moisture conditions, such testing provides a useful means of building up knowledge of appropriate pavement design criteria for local soils under the locally prevailing climatic conditions.

In situ CBR measurements of heavy clays usually correspond quite closely with the results of laboratory CBR tests on the soil at the same density and moisture content. With soils of lower cohesion, which rely partly for their stability on internal friction, the confining effect of the laboratory test mould is an important influence on the test results obtained, and there is less correspondence between the results of laboratory and in situ tests. This correspondence can be improved by loading the test surface with annular metal rings to simulate the confining effect of a pavement, but in situ CBR testing is not a suitable means for checking compliance with a specification during construction.

2.3.9 *Consolidation* BS 1377, Part 6.

Consolidation involves the squeezing out of water from saturated soils, as distinct from compaction, which essentially involves the reduction of air voids in unsaturated soils. Both processes have the same objective, to reduce further settlement of the soils under the load of civil engineering structures (compaction may have the additional purpose of increasing the bearing capacity of the soil immediately under the structure).

Unconsolidated soils are widely distributed in deltaic areas and coastal regions in tropical countries. They are usually silty clays or clayey silts and they differ little from the soils occurring in similar situations in other parts of the world, with the possible exception that the great rivers, discharging into tropical seas, can produce deposits of greater depth.

Consolidation testing is employed to estimate the extent to which these soils will settle under the load of an embankment or other structure and to estimate how the rate of loading should be controlled so that the soil consolidates evenly without being displaced laterally, as will happen if the loading produces excessive water pressures in the soil. On road embankments, the objective is to control the rate of construction so that the consolidation of the underlying soil is substantially complete when the embankment reaches its full height.

The test method derives from the theory of consolidation produced by Terzaghi. This theory states that if the load on these saturated soils is increased at a uniform rate, the settlement that occurs when loading is stopped is equal to the settlement that would have occurred at half the time had the final loading been applied instantaneously at the beginning.

There is a variety of forms of consolidation test employing different forms of loading. In its simplest form, loading is applied vertically on to a cylindrical specimen mounted between porous plates and a typical consolidation cell is shown in Fig. 2.14. As shown in Fig. 2.15, (TRRL, (1976)), the test can give a remarkably precise indication of the total amount of consolidation to be expected under field conditions, but it usually underestimates the rate at which consolidation occurs in the field.

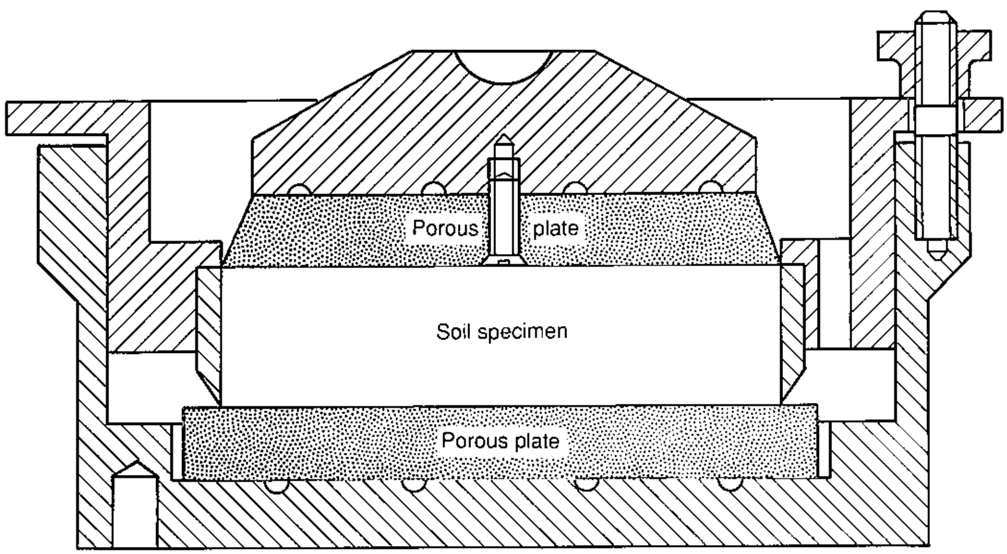


Fig.2.14 A typical consolidation cell.

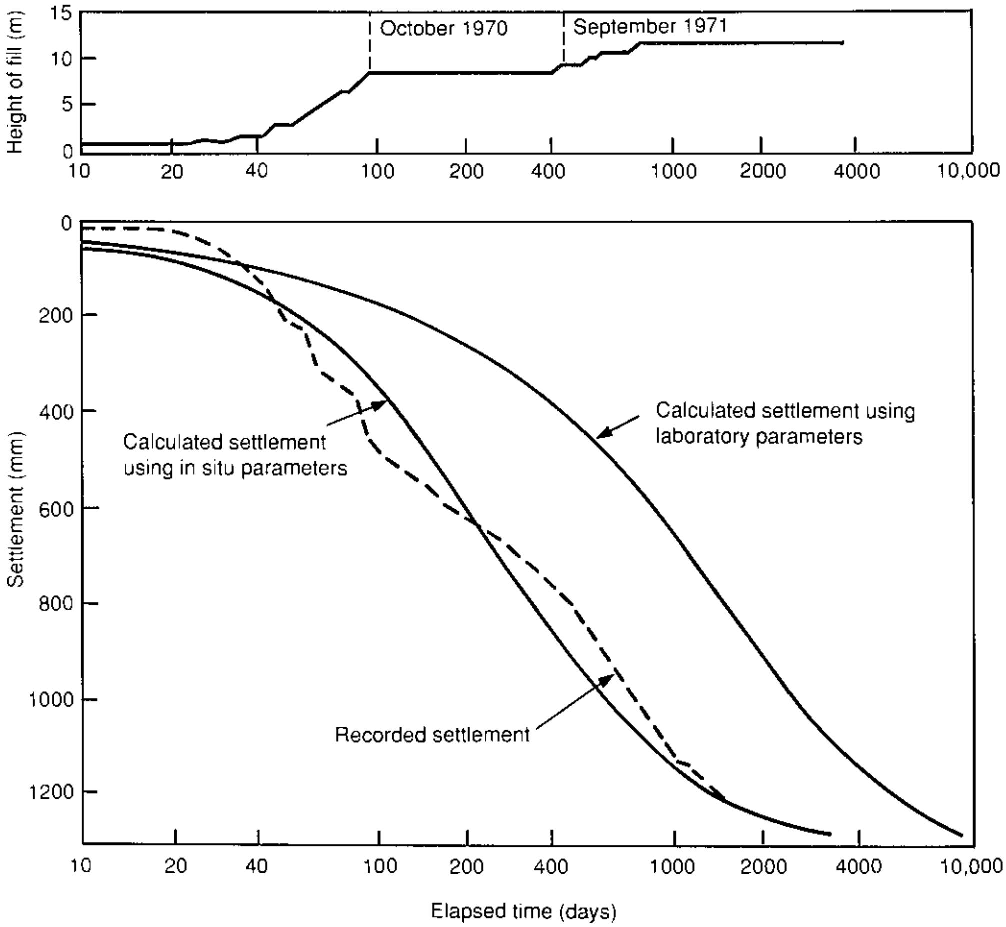


Fig.2.15 Comparison of measured and calculated settlement.

This rate is determined by the rate at which water can be safely dispersed without building up dangerous excess water pressures in the soil under an embankment. Frequently, horizontal sand lenses occur in these soils, remnants of exceptional floods in the past. These lenses provide a path through which water can be dispersed more rapidly than through the surrounding soils. Such paths can be provided artificially by vertical sand or wick drains.

Consolidation testing is undertaken as part of survey and design whenever unconsolidated soils are encountered. The results will appear in contract documents as limitations on the form and speed of embankment construction and in the possible provision of vertical sand or wick drains so that the rate of consolidation can be accelerated. More information is given on construction over these soils in Chapter 11.

2.3.10 *Triaxial Compressive Strength* BS 1377; ASTM D4767.

Triaxial testing is used extensively in research to define the characteristics of soils, rock and bituminous road mixtures; consequently there are many forms of apparatus and methods for these purposes. In routine design, the test is likely to be used only when it is necessary to examine the structural strength of soils as foundations for structures or for use in high embankments for roads and dams. For this purpose a standard testing method is used for determining the compressive strength of a specimen of saturated cohesive soil under conditions in which the cell pressure is maintained constant and there is no change in the total water content of the specimen. This test is rarely used in road design, being needed only when plastic soils are used for constructing embankments which will contain soil in a saturated condition, and in detailed studies of slope stability problems.

2.3.11 Other soil tests

There are other tests likely to be used for soils in survey, design and control of construction processes. These include the determination of the moisture content and organic matter content of soils, the determination of the sulphate content of soil water, field and laboratory tests to determine the strength and compressive strength of cohesive soils, and the penetration resistance of both cohesive and non-cohesive soils. Procedures for these tests are defined in BS 1377 and in the equivalent ASTM and AASHTO standards. Whenever any of these tests are used, an appropriate standard testing procedure should always be employed.

2.4 Future developments

Of the test methods reviewed in this book, it is with the testing of soils that changes in testing procedure are most likely to be introduced as knowledge increases on their engineering interpretation and on how to improve the consistency of results. It is particularly important to make sure that the most up-to-date editions of the Standards are available to those responsible for survey, design and control of construction processes.

Some of these tests will be carried out in the field during the survey e.g. measurements of field densities of soils and simple tests to evaluate the extent and likely quality of gravel deposits. Most of the tests will be done on samples brought back for evaluation in the materials laboratory. These include compaction tests to determine the densities to which the soils are to be compacted, their bearing capacity when so compacted, tests to evaluate the strength of materials available for the road bases and to evaluate the quality of rocks available for use in making the road surfacing. Where the roads run over unconsolidated saturated soils such as the silty clays common in coastal areas of many tropical countries, a special testing programme will be necessary to determine appropriate construction techniques.

Many tests have been evolved for these purposes during the last 50 years. Some have been discarded, either because their results are of poor engineering significance, or because simpler, less laborious testing techniques have evolved. We now seem to be settling into a era in which a particular range of tests is accepted all over the world, in survey, in design and in the control of construction.

This approach to the structural design of roads, through soil testing, has worked reasonably well, but it has other weaknesses in addition to the foibles of some of the test methods. Experience in interpreting the engineering significance of the test results has been built up, primarily, in temperate regions, and this interpretation often needs to be adjusted for use in other climates. For some aspects of field performance, resistance to erosion for instance, there is no commonly accepted testing procedure. When surveying the line of a new road, it is often impossible to sample and test the soils to the extent necessary to provide a complete picture of the engineering properties of all the soils which occur along the route.

2.5 Terrain evaluation

The weaknesses described above have prompted development in a complementary direction, one in which more use is made of a recognition of the way in which different soils have evolved and how this evolution has influenced the nature and the engineering properties of the soil. It is based on the premise that wherever in the world the basic geology, topography and climate are similar, then similar types of soils will develop. With a knowledge of the soil forming processes at work in a particular area, it is possible to locate and map the boundaries between the different types of soil and to build up knowledge of their engineering properties and uses. Indeed, it opens up the prospect of transferring this knowledge between different parts of the world where similar soils occur.

This approach through terrain evaluation is proving of particular value in tropical and subtropical areas. The weathering of rock to form soil proceeds much more rapidly in hot humid areas than in more temperate zones. The rocks are weathered to a much greater depth and many of the soils and surface rocks are still undergoing chemical alteration which changes their physical characteristics and affects the way they behave as foundations for engineering structures.

In temperate regions most of the soils are transported by water, by glacial action and, more rarely, by air. They have been heavily worked during such transport. Because of this and because further weathering is very slow indeed in cool climates, most soils have reached a stable condition, both chemically and physically. In the tropics most soils are residual i.e. they have been formed, and are often still being formed, over or in close proximity to the rocks from which they originate. Some of them are far from stable, both physically and chemically, and this means that their evaluation by physical tests must be supplemented by a knowledge of their idiosyncrasies. The knowledge of the manner in which they occur in a particular locality can be of great value in making the work of soil survey easier and more useful.

The wide distributions of these tropical residual soils is shown in Fig. 2.16 which comes from a recent detailed review of tropical residual soils prepared and published by the British Geological Society (1990). This review describes the nature and occurrence of the different residual soils and gives valuable information on how their peculiarities influence their testing and use in civil engineering. Each type of residual soil has well defined characteristics deriving from the nature of the parent rock and the climate prevailing in recent geological time. Thus, whenever

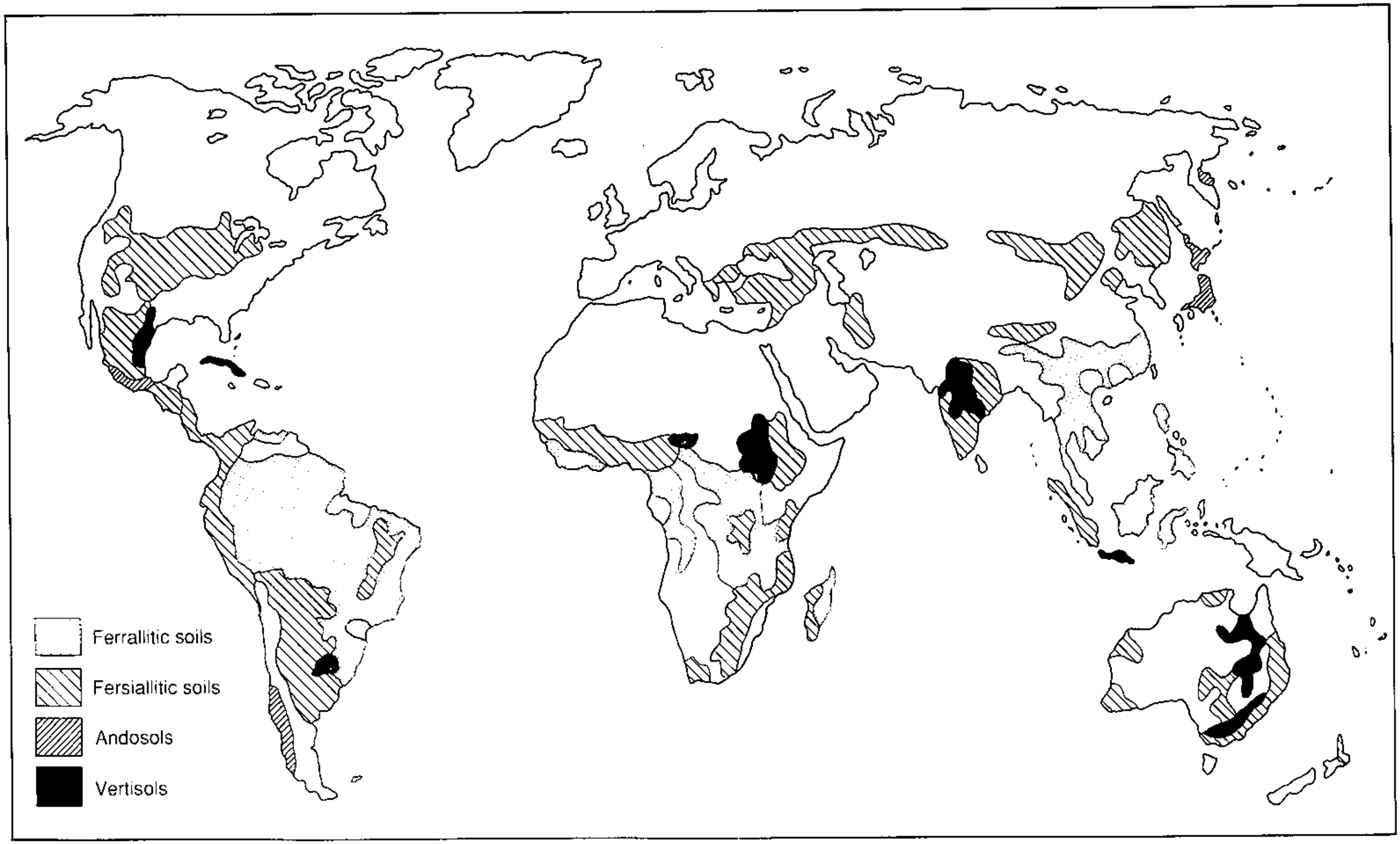


Fig.2.16 *Distribution of the principal types of tropical residual soils.*

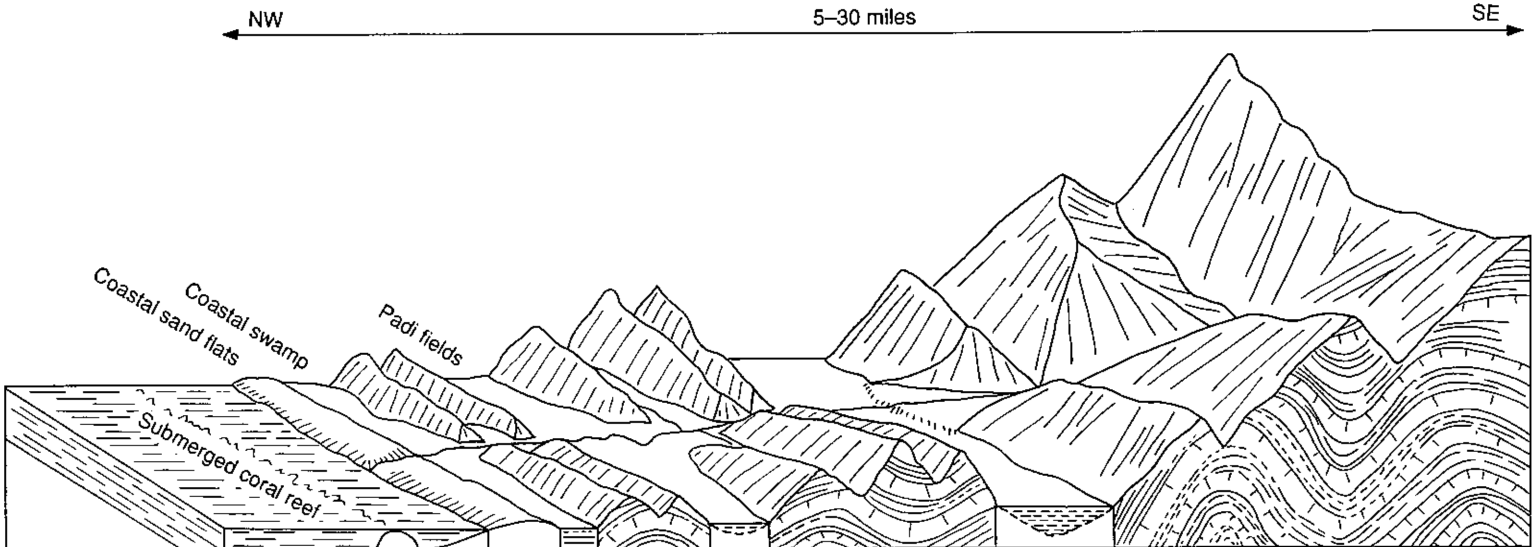
similar geological and climatic conditions occur, the same types of residual soil are evolving, but sometimes at different stages of development.

Basic to the understanding of this evolution is the catena sequence. In its simplest form, this is merely the process by which the more soluble products of rock weathering are dissolved by rain and are carried away down the catena whilst the less soluble remain behind. The solubility of the weathered materials is affected by the acidity of the water. Even without the assistance of acids from industrial emissions, rain is slightly acid from dissolved carbon dioxide. Such acid rain will mobilise the relatively insoluble iron and aluminium oxides, but they do not travel far. In hot climates, organic material rapidly decomposes and is washed away. Ground water is often neutral or alkaline, causing the metallic oxides to be redeposited. Silica is also relatively insoluble but can be mobilised to combine with other weathering products to form clay or, more rarely, to be deposited as silica gel forming the silcretes found in some desert areas. The salts of calcium, magnesium, potassium and sodium travel further, the chlorides going all the way to the sea whilst others either combine to form clay minerals or are deposited in dry areas as calcrete, gypsum etc.

Land-form is also a potent factor in soil formation. On steep slopes, rock detritus accumulates in deposits at the foot of the slopes and minerals moving in solution may accumulate in these deposits. Areas of impeded drainage promote the formation of particular types of clay as well as providing a site for the redeposition of dissolved minerals, a classic example being the extensive deposits of black cotton soil (montmorillonitic clay) underlain by jigilin (nodules of calcium carbonate) in the area around Lake Chad. Another illustration of the effect of land-form comes from a comparison of the soil profiles developed on granite rock in Hong Kong, described by Lumb (1962), and in the sand-veldt areas of Zimbabwe. A similar sequence of quartz particles underlain by leached clay occurs in both places. In Hong Kong, the parent rock was shattered by uplift and the weathering penetrates to a depth of 50 metres or more, whilst in Zimbabwe, the granite is in the form of a solid horizontal batholith and the soil profile is compressed into a depth of 2 - 4 metres from the surface.

One purpose of the two examples given above is to illustrate that, although the mechanisms of soil formation may appear complex to those without training in pedology, the application of the knowledge to engineering is often quite simple. In any one region there are likely to be only a few land systems, perhaps only one or two, certainly no more than a dozen. Each has its own particular geology and land-form and, within each, the residual soils will occur in the same sequence. Once these land systems have been identified, the door opens on a method of locating the different soils of a region and of accumulating knowledge of their engineering properties and uses. The work of ground survey can be considerably simplified using aerial survey, not only to define the topography but also to indicate the boundaries of the different soils and the likely location of road making gravels and quarry sites.

The Overseas Unit of the Transport Research Laboratory has been active for some years in developing this approach. It was first motivated by the opportunity it offers for the use of aerial photographs in indicating both the nature and the boundaries of different surface deposits. More recently it was prompted by the deeper understanding that the method can provide for engineers of the nature and engineering uses of the soils they encounter in the regions where they work. An early example, described by Clare and Beaven (1965), comes from Sabah. Here, as in many other Pacific islands, steep mountains of volcanic and metamorphic rock lie behind a narrow coastal plain. This terrain is illustrated in Fig. 2.17 together with notes on the engineering characteristics



		Beach sand	Peat	Sandstone and shale	Alluvium and peat	Sandstone and shale	Alluvium and peat	Sandstone and shale
Topography		Flat	Flat	Steep slopes 20°–30°	Flat	Steep slopes 20°–30°	Flat	Very steep slopes 30°–45°
Drainage conditions		Well drained but high water-table	Very high water-table	High run-off and good drainage	High water table particulary through padi fields	High run-off good drainage	High water-table	High run-off
Available road base material		Coral		Crushed rock		Crushed rock	River gravel	Crushed rock
Maintenance problems		Occasional flooding	Floods shifting found [illegible]	Scour of drains & culverts washouts and landslips	Flooding	Scour of drains washouts and landslips	Flooding	Scour, washout, landslips

Fig.2.17 *Geology, land-form and road making characteristics on the western coast of Sabah.*

of different facets of the terrain. The methodology for establishing such a system of land-form classification and for characterising the soils within each land-form have been described by Cook and Newill (1988), by Beaven and Lawrance (1982) and by Lawrance *et al* (in press). It has been used in many countries, including the Caribbean islands, Colombia (Heath *et al* (1978)), Nigeria, Botswana (Lawrance and Toole (1984)) Ethiopia, Nepal and mainland Malaysia (Lawrance (1977) and (1978)).

In all tropical countries, one of the most useful tasks that Government Materials Laboratories and University Civil Engineering Departments can undertake is to classify the land-forms within their boundaries and to characterise the form of occurrence and engineering properties of the soils and surface rocks within each land-form. Thus they can provide a basis for teaching students of civil engineering and of agriculture about the material resources of their own countries in a way which should fire their imaginations and much increase their usefulness.

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3 Particular tropical soils

There are several well recognised groups of soil that merit mention because of their wide occurrence, their special properties and their uses in road engineering. These include concretionary soils such as laterites, which are a main source of material for building gravel roads and making bases for bituminous-surfaced roads, volcanic soils, which often require special treatment because of the unusual clay minerals they contain, expansive soils e.g. black cotton soil, which can present special engineering problems, and desert sands.

3.1 Concretionary soils

3.1.1 Laterites

By far the most widely occurring concretionary soils are the laterites. The term laterite has been put to diverse usage since it was first coined by Buchanan, and pedologists have added to the engineer's confusion by adding supplementary terms latosol, ferrisol etc. It is therefore worthwhile defining the meaning attached to the term laterite in this book. The oxides of iron and aluminium are amongst the least soluble components of weathered rock, remaining behind when other more soluble products have been leached away. They can, however, be mobilised in acid rain and subsequently redeposited. Aluminium occurs in concentrated form as bauxite and, more widely, as a constituent of many clay minerals. The iron also occurs in concentrated form and also as a constituent of many soils. It is the soils and rocks containing iron oxides as a major constituent in concentrated form that are given the designation laterite in this book. As shown in Fig. 3.1, laterite soils occur widely throughout the tropics. Indeed, the iron-rich rocks and soils mined as iron ore in temperate zones are likely to be remnants of the time when a tropical climate prevailed.

Such laterite occurs in three distinct forms. In Southern Asia it occurs widely with the iron oxides as a major constituent in surface deposits of clayey soil. These clays are quite soft when first exposed and they have the remarkable property of hardening irreversibly on exposure to the air. Blocks of clay, easily excavated and shaped by hand, harden after exposure for a few weeks into pieces of masonry which are extremely hard and durable. The Khmers used such masonry blocks in building their successive capital cities as they moved south some 1000 years ago, through present day Thailand culminating at Ankhor Wat. In markets in northern Thailand, such blocks are still exposed for sale. This is the original laterite as identified and named by Buchanan (1807). This hardening is associated with oxidation from ferrous to ferric oxides.

Whenever lateritic clays are exposed on the ground surface in areas with a hot dry season, they harden to produce a cuirasse of hard rock which is brick red in colour. This second form of laterite occurs in Southern Asia, in Africa and Australia. (Soft lateritic clays have not been reported from either Africa or Australia and it seems likely that climatic conditions have been such that the

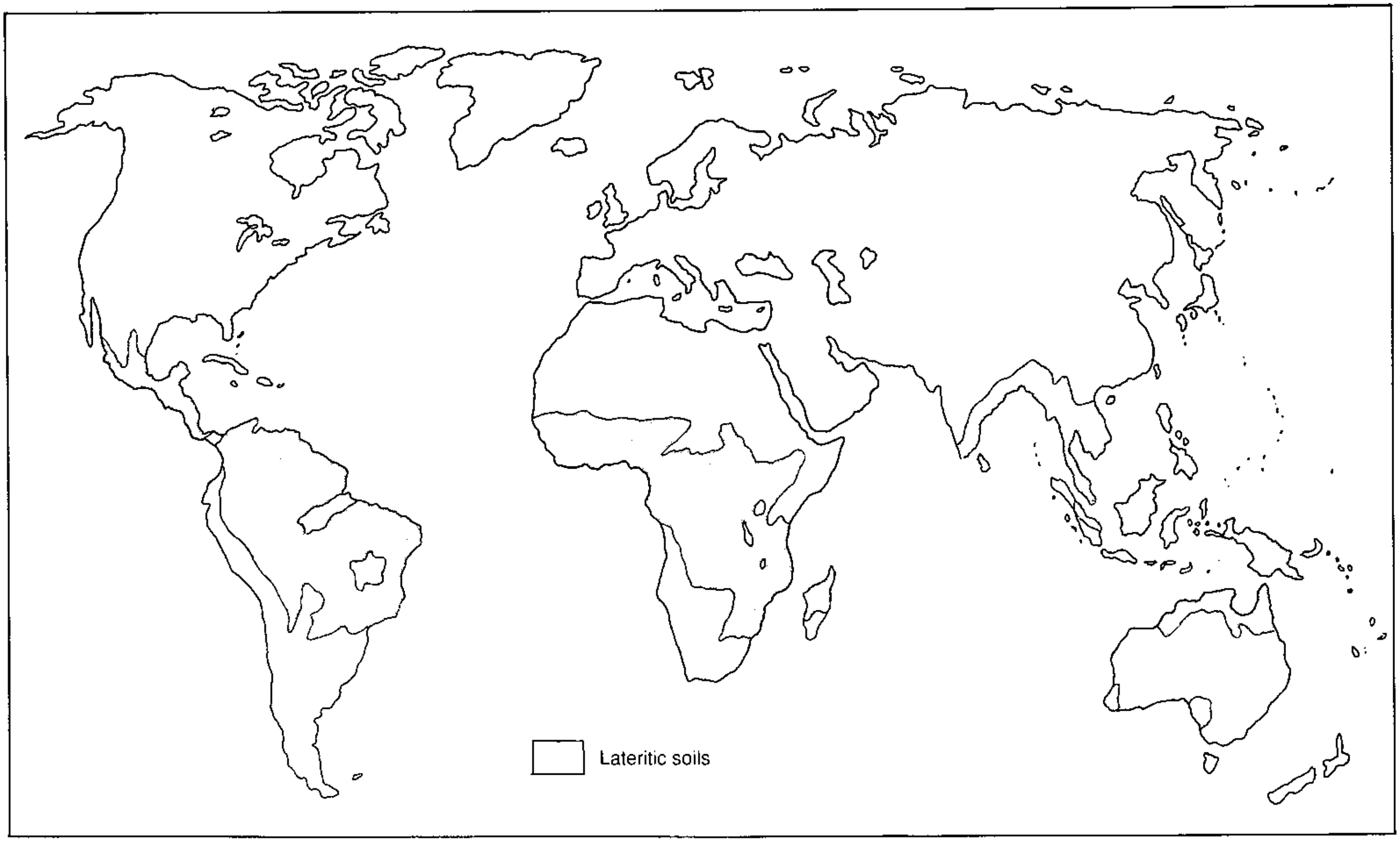

Fig.3.1 *Worldwide distribution of lateritic soils. (Charman, 1988, CIRIA Special Publication 47)*

hardening process has been long completed in these two continents.) These hard lateritic reefs occur widely in West and Southern Africa. They can be found capping hills, with the softer rock below weathering away to produce typical mesa formations. Conakry, the capital of Guinea, is built on a lateritic reef running into the sea. This gives rather a formidable appearance to the shore line but it is ore of a very high iron content. These reef laterites are of little interest to the road builder. The rock is usually extremely hard, heavy and abrasive and it soon wrecks quarry equipment brought in to exploit it.

It is the third form of laterite, the lateritic gravels, which are of value in building gravel roads and bases for bituminous roads (Charman (1988)). They occur in almost all tropical countries and are found in isolated deposits, often several hectares in area, bedding horizontally and in thicknesses generally between 1 and 5 metres. They are usually oval or crescent shaped in plan. Although they must have originated in the upper slopes of the catena, subsequent erosion of the surroundings may conceal their origins. Typically, they stand out as low humps in the surrounding countryside. They support little or no vegetation. Thus, in any particular region, it is possible to characterise both the topography and vegetation typical of the deposits and to use aerial photographs to identify likely locations for ground survey (see Dowling (1966) and also Schofield (1970)).

Lateritic gravels occur in specific localities in most African countries south of the Sahara, in South America, in Southern Asia and in Australia. In Uganda they are particularly abundant and of good road making quality. In Kenya and in Malawi there are signs that deposits within easy reach of road building activities are becoming depleted and measures are being taken to limit their use in building and resurfacing gravel roads. When used as a roadbase under a bituminous surfacing, gravels remain in place for the life of the road, but when used as a gravel running surface, the material is lost within a few years by the erosive effects of traffic, wind and rain.

The most remarkable characteristic of these gravels is that they have a characteristic particle size distribution wherever they occur. Texturally they are usually clayey gravels. They consist of particles (pisoliths) between 6 and 20 millimetres in size embedded in a matrix of fine-textured soil. These pisoliths are spherical in shape and are local concentrations of iron oxide in the soil. A typical exposed profile is shown in Plate 3.1 and their typical gap-graded particle distribution is shown by the gradings of lateritic gravels from different parts of the world illustrated in Fig. 3.2.

These lateritic gravels have been the subject of many studies and a selection of publications describing their formation and engineering use is included amongst the references at the end of this chapter. There is common agreement that they generally derive from basic rocks and that they develop only in hot climates with marked wet and dry seasons, but there is, as yet, no satisfactory explanation for the ubiquitous occurrence of the pisoliths of similar size and spherical shape. One suggestion is that they represent an intermediate stage in the development of the reef laterites but this is not fully convincing since the lateritic clays of Southern Asia harden quickly and directly to form solid blocks of lateritic rock. Lateritic gravels are not found in the huge masses characteristic of the lateritic cuirasses. When broken down, some pisoliths reveal a concentric structure which suggests that they may have been formed by the precipitation of iron oxides round a small particle of alkaline material. But other pisoliths appear to be quite amorphous.

The pisoliths are generally very hard and tough but in some areas, for example in western Nigeria and north east Zambia, there are deposits with quite weak nodules, so weak that they crumble

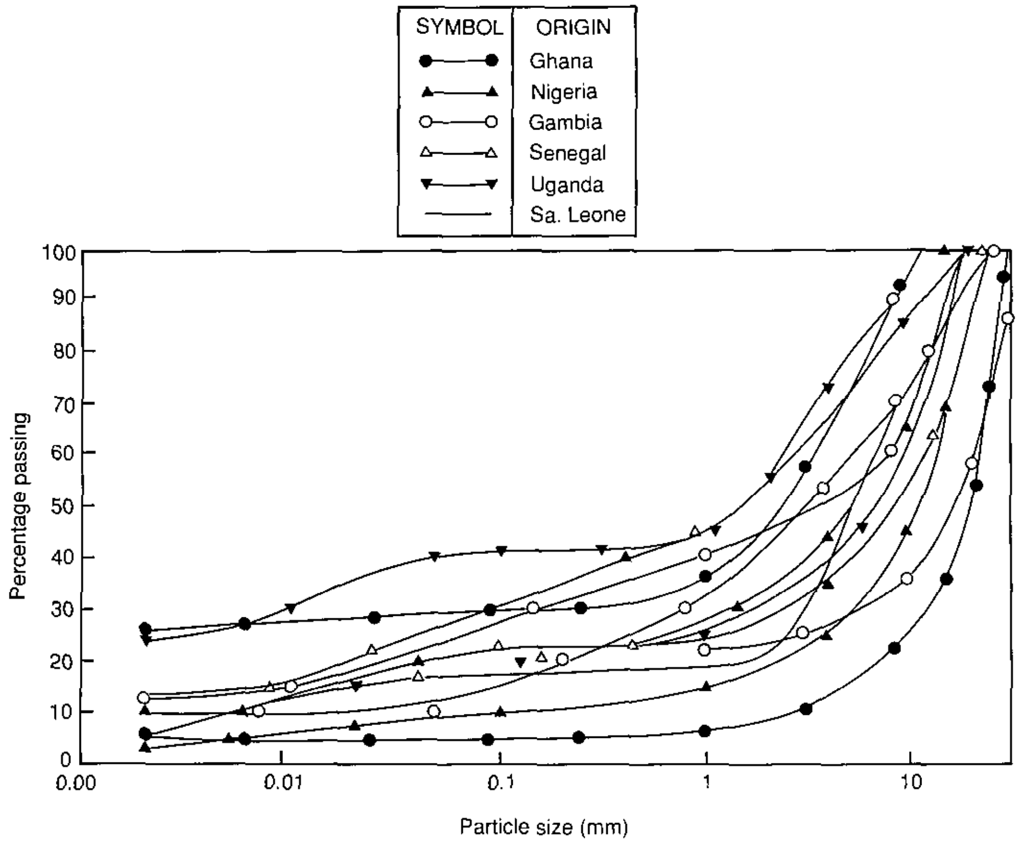

Fig.3.2 *Typical gap-grading of lateritic gravels from different sources.*

when roughly handled. This variation in the hardness of the pisoliths is associated with the degree of laterisation and is reflected in their specific gravity, as shown in Fig. 3.3. Hand inspection is likely to be sufficient to reject gravels with very weak nodules for road making. When a more numerate method of evaluation is needed e.g. in testing gravels offered by contractors, the Aggregate Impact Test or the Los Angeles Abrasion Test (Chapter 4) are convenient. Gravels suitable as running surfacings on well trafficked roads or as bases under bituminous surfacings should have values less than 45 per cent in either test. The nodules are at their weakest when they are wet and the tests should be done on nodules which have been soaked in water for 24 hours.

Even when the nodules are quite hard, some change in the structure of the gravels is likely during handling and compaction and when they are wetted. This complicates evaluation by physical testing. An indication of their susceptibility to mechanical breakdown can be obtained by undertaking particle size analysis on samples before and after they have been subjected to laboratory compaction tests. The nodules are likely to be of higher specific gravity than the clayey fines. Where this difference is large, typically 3.5 compared with 2.6, an accurate particle size

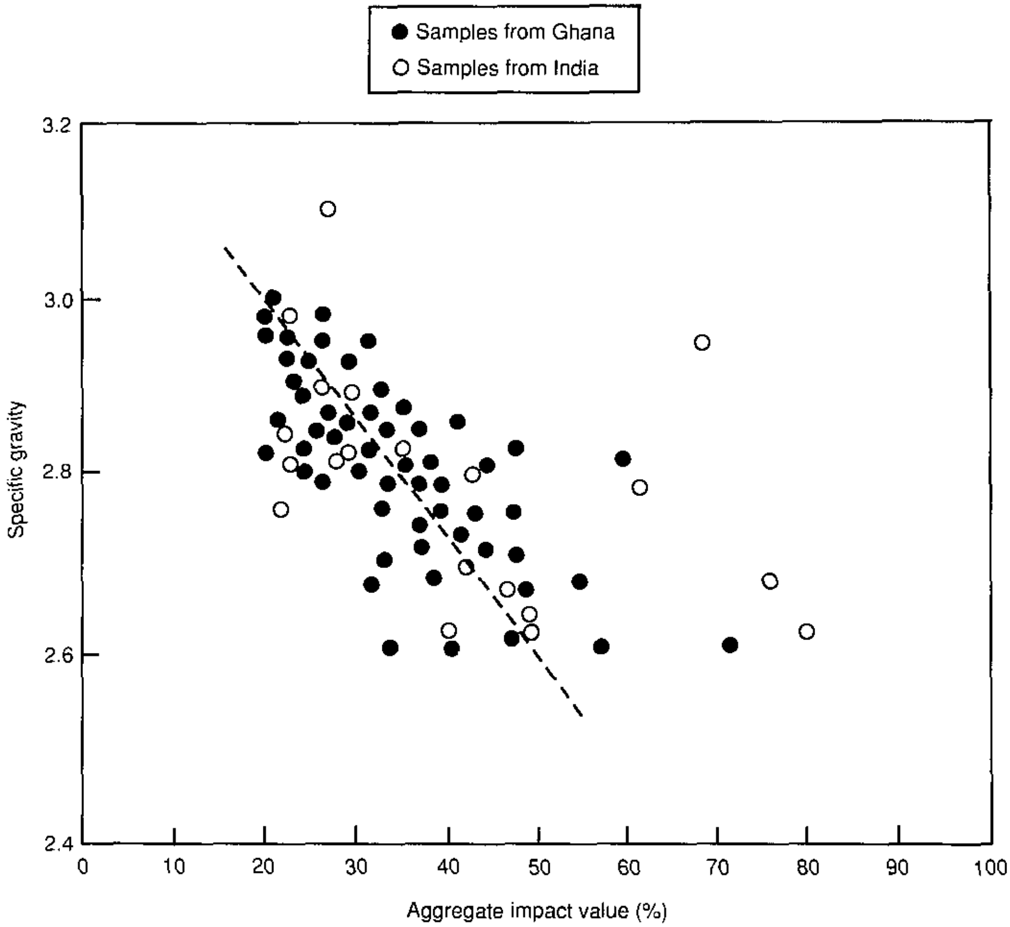

Fig.3.3 *Relationship between specific gravity and Aggregate Impact Value for typical laterite rocks. (Charman, 1988. CIRIA Special Publication 47)*

analysis requires modification to give the proportions on the different sieve sizes by volume rather than by mass (see Fig. 2.2).

There have been suggestions for many years that lateritic gravels can harden further when built into the road and that this hardening is similar to that observed on exposure of lateritic clays to the atmosphere. Hardening certainly occurs but it is more likely to be due to a combination of other causes. On gravel roads, for example, a reduction in the clay content in the dust raised by passing vehicles, combined with further compaction by traffic, results in 'hardening' which is further accentuated by the suction developed by the remaining clay fraction as the soil dries out. In Africa there were many failures of bitumen surfaced roads with gravel bases before the need for thorough compaction of gravel bases was properly appreciated. These were usually associated with the penetration of water from the sides or through a defective bituminous surfacing. Their vulnerability to the weakening effect of penetrating water can be eliminated by stabilisation with quite small amounts of cement or lime (see Chapter 8) but Grace *et al* (1985) have demonstrated that, with careful engineering, lateritic gravels can be used without such stabilisation to make bases which perform well under bitumen surface dressings on roads carrying typical rural traffic.

3.1.2 Calcretes and silcretes

These are concretionary materials found at particular localities in many deserts, usually not far from the rocks from which their constituent materials were derived. It is the carbonates of calcium and magnesium that have been mobilised and redeposited to form calcretes and it is quartz which forms silcretes. The calcrete deposits usually contain quartz but the silcretes are almost entirely of quartz. Calcrete and silcrete are the terms used to designate these materials in Southern Africa but they go by other names in other parts of the world. For instance, the sabkha which occurs in the coastal regions of the Arab peninsular is a salt bearing soil containing concretionary calcium carbonate (see Section 3.4).

In the Kalahari desert, both calcrete and silcrete occur as surface deposits where the wind-blown sand overlies rocks in which a well defined river drainage system had developed and where there is a short but well defined rainy season. They occur in low lying areas, frequently in pans i.e. shallow round depressions, often over 500 meters in diameter. A raised platform in the middle of these pans is an indication of calcrete in massive form resembling soft limestone (see Plate 3.2). Calcrete can also occur as nodular gravel, as calcified sand, or as fine powder. Neither the massive calcrete nor the nodules are very hard and they would not be used for road making in areas where stronger materials are readily available. Lawrence and Toole (1984) and Lionjanga *et al* (1987) have described how calcretes can be located and used to make quite satisfactory road bases for bituminous-surfaced roads in Botswana.

Silcretes are less frequent than calcretes but they, too, are found in pans. They can resemble a soft sandstone but also occur as nodules or as a fine white powder, amorphous opaline silica. The strength of weak calcretes and silcretes can be improved by stabilisation with cement or lime, but beware with the silcrete powder. During an early attempt to stabilise a mixture of this powder and sand with lime, the mixture expanded out of the laboratory test moulds during curing. This was an early and extreme example of the alkali-silica reaction (ASR) which is wreaking damage in many countries to structural concrete made with aggregates containing opaline silica.

3.2 Other gravels

3.2.1 Alluvial gravels

Alluvial gravels occur in the upper reaches of most rivers. They consist of rounded boulders and pebbles of hard rock, the weaker parts of the parent rock having been ground to sand and silt and carried further down river towards the sea. They are usually of igneous or harder metamorphic rock. Well sorted deposits of river borne gravels and sands can often be found which are suitable

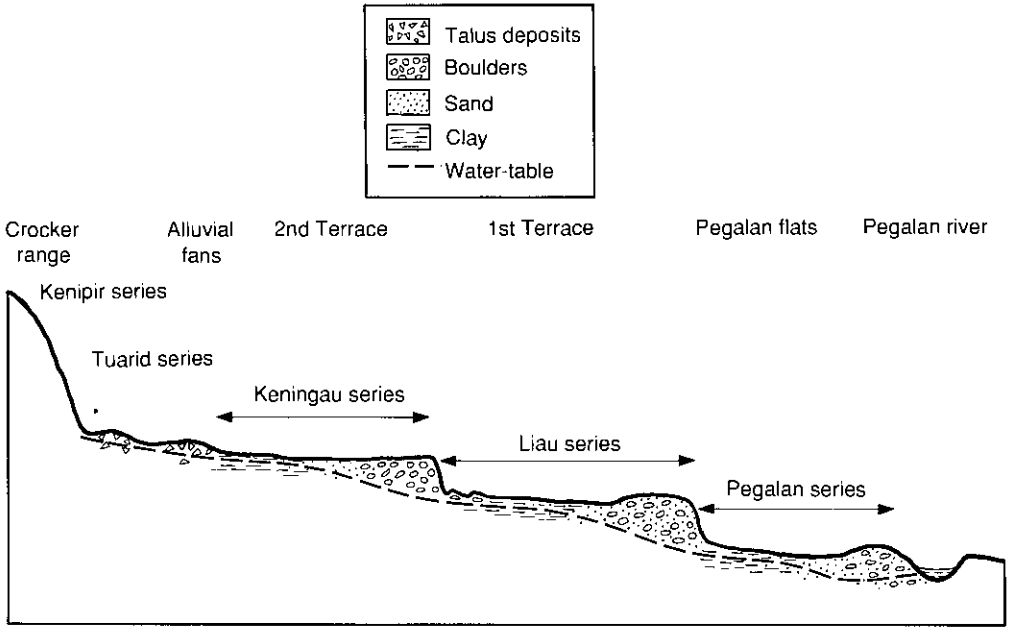

Fig.3.4 Diagrammatic section of gravel terraces on the Keningan plain, Sabah.

for making concrete. In the absence of rock deposits for quarrying, the gravels can be crushed to produce aggregates for bituminous surfacings and road bases.

3.2.2 Terrace gravels

Terrace gravels feature more commonly in northern latitudes, laid down and often cut through by the flooding which followed successive periods of glaciation. They occur in tropical regions where geological uplift or changes in sea level have produced periods of dramatic increase in river flow. Clare and Beaven (1964) have noted the typical land-form containing terrace gravels in Sabah (Fig. 3.4). Such gravels, when crushed, have been used in making bases for roads and airfields in Sabah, Brunei and Sarawak and it is likely that they are available for exploitation in many regions where flood plains abut on to precipitous mountain slopes. They are usually mixtures of coarse stone particles, sand, silt and clay (Plate 3.3). In hot humid climates it is likely that such gravels originate from rock that is already weathered. They have not travelled far and, though some sorting into deposits of different sizes may have occurred, they may not have been subject to the abrasive action of transport by water which removes weaker particles. It is wise, therefore, to check whether the weathering of the coarse particles has reached a stage when further decomposition can occur rapidly, weakening the road bases in which they are used (See Chapter 4).

3.2.3 Residual quartzitic gravels

These derive from the in situ weathering of granite and other coarse-grained acidic rocks or from the weathering of rock debris which has accumulated on the lower slopes of formations of these rocks. Such rocks contain a high proportion of quartz, which is more resistant to weathering than the other minerals they contain. As these latter decompose, they are carried down the soil profile and build up a clay enriched zone at some depth below the surface. Such quartzitic gravels, formed from rock debris, can contain iron oxides leached from above, thereby producing the laterised quartz gravels that occur in many parts of Africa. Some of the quartz is also mobilised, as is evident from the many soil profiles that display a marker line of quartz pebbles, sometimes up to 50 millimetres in diameter, immediately above the zone of clay accumulation.

As with lateritic gravels, deposits can often be located by the sparse vegetation they support and by their obtrusion in small hills above the surrounding countryside. Usually the maximum grain size is not large, often less than 30 millimetres. Their grading can vary considerably, as can their clay content. They are usually stratified and, in exploiting a deposit, it is important to survey both the depth and area of the deposit and to keep a constant watch during extraction to make sure that underlying clayey soil is not included. As with lateritic gravels, it is desirable to bulldoze the gravel into stockpiles which can be regularly sampled to make sure that quality is maintained. Standard testing procedures are appropriate with quartzitic gravels. Where they contain both quartz pebbles and lateritic pisoliths, it is important to indicate the relative proportions of the two materials in the coarse fraction and, where the proportion of lateritic pisoliths exceeds 20 per cent, to evaluate their strength and resistance to mechanical breakdown.

In the sand-veldt areas of Zimbabwe, gently undulating plains derived from the in situ weathering of granite, quartzitic gravel deposits are frequently found on higher ground. Over much of these areas the leached clay forms an impermeable hardpan at a depth of 2-4 meters below the ground surface. This can produce perched water tables in the wet season, a source of pavement weakness if they are not identified and dealt with during construction (see Chapter 11).

3.2.4 Volcanic scoria

These are commonly found in areas of recent volcanic activity. They are vesicular cinders, resembling in appearance but usually weaker than the blast-furnace slags which are excellent road making materials in many industrialised countries. Such scoria vary in size and in hardness. They can be used as dug to make lightly trafficked gravel roads, as reported from Ethiopia by Newill and Aklilu (1980). Plate 3.4 shows the terrain in which volcanic scoria are found in Ethiopia. Experience so far suggests that deposits of such material are not sufficiently extensive nor uniform in quality for use in making the bases and surfacings of more heavily trafficked roads but, where they do occur, they are easily worked and well suited for use in building local roads with a minimum of equipment.

3.3 Clay soils

3.3.1. Volcanic clays

The residual clayey soils that develop on volcanic lava flows are usually very fertile, supporting intense agriculture. They present no unusual difficulties to the road builder except that, because of intensive roadside development, he is likely to meet resistance when he seeks to acquire land for necessary road widening as traffic increases. In Java, for instance, many of the older main roads are closely hedged in between buildings in continuous development between towns and villages. In Mauritius, sugar cane threatens to occupy the roadside ditches.

The soils that develop on volcanic ash are a different matter. As the ash weathers, it retains much of the loose structure it had when originally deposited and, in hot humid climates, a sequence of clays develop which have rather unusual properties. This sequence follows a particular pattern, first allophane is formed which has an amorphous gel-like structure, followed, in climates in which the annual cycle includes hot dry seasons, by halloysite and metahalloysite. These latter have the unusual feature that the clay particles are scroll-like rather than the flat plate-like particles in other clays (Fig. 3.5). The properties and uses of these soils are comprehensively described in the review of tropical residual soils published by the Geological Society (1990).

The allophane stage can persist for long periods in areas of persistent high humidity. Such clays retain large quantities of water, often more than 200 per cent of their own dry weight. Their structure is fragile and they can collapse catastrophically when loaded. Such clays occur in many volcanic areas, notably in Japan, Hawaii and the West Indies. The effect of dry heat on allophanic soils has been dramatically illustrated by tests on an allophanic clay from the Pacific Island of Vanuatu. This soil had a natural moisture content of about 250 per cent. As is shown in Fig. 2.5, the soil changed, on heating, from a highly plastic clay, developing small concretions to the extent that it became a non-plastic sand. Further examples of the effects of drying and heating on the plasticity characteristics of allophanic and halloysitic clays are given in Table 2.5.

It has been suggested that allophanic clays might be transformed into useful engineering soils by drying them either naturally or by the use of heaters. But these soils are so highly sensitive to disturbance that they are best avoided altogether and very great care is necessary when working in their vicinity to avoid interfering with any natural drainage paths that are present and, above all, not to subject them to extraneous loads which are likely to produce catastrophic collapse.

Soils that have developed to the halloysite and meta-halloysite stages have rather different properties. Typical of them are the red-coffee soils of East Africa. The curled and twisted clay particles help to retain the open surface of the volcanic ash and this structure may be reinforced by iron oxides taken into solution and redeposited around the points of contact. The change from halloysite to meta-halloysite appears to be the result of persistent dry heat. Both clay minerals are likely to be present in such soils and it would be difficult to distinguish any difference in the engineering performance of soils containing a preponderance of either mineral. These soils have low field densities, often in the region of 1.0 Mg/m^3. They are free draining and they retain their structure unless they are heavily worked.

It is the effect of working these soils which is particularly significant, both in testing them and

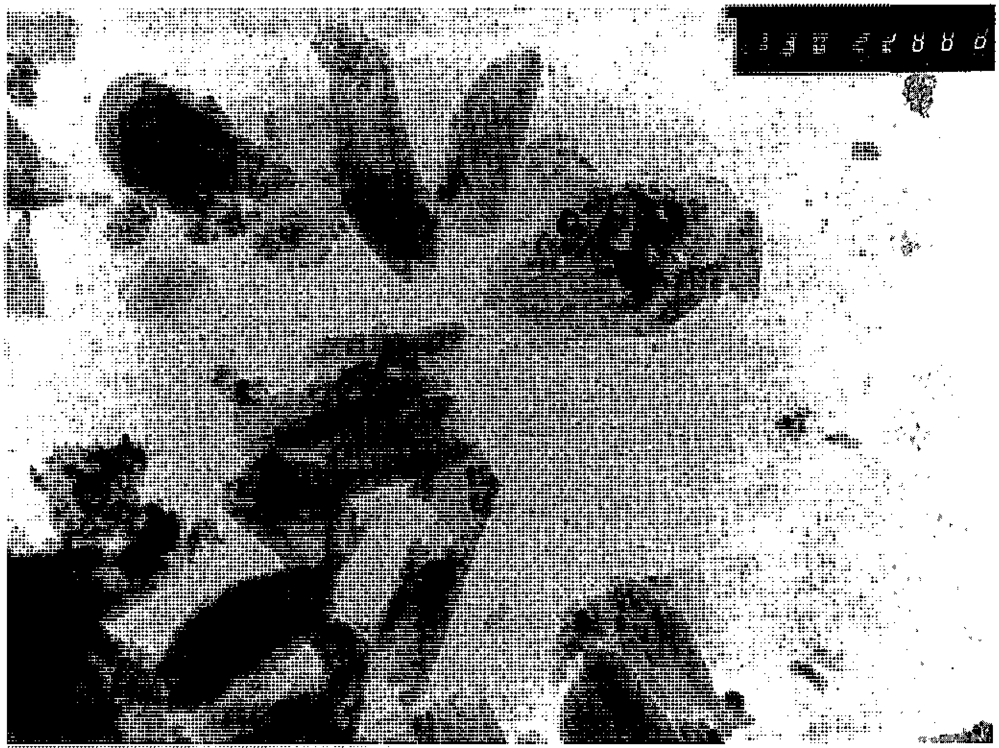

Fig.3.5 Photomicrograph of metahalloysite showing structure (Courtesy British Geological Survey).

on their engineering use in the field. The effect of working is to break down their structure irreversibly so that they become more clayey than they appear in nature. An early example occurred during the construction of the Sasumua dam in Kenya where Terzaghi was called in to investigate contractual problems deriving from the classification of halloysite soil as a heavy clay (Terzaghi (1958)).

Quite good tracks and lightly trafficked roads can be made over these soils since their structure and free draining properties make them less susceptible to rutting than other clayey soils. They become rather slippery when wet as a result of water being squeezed out of the soil under the contact pressures between tyre and road surface. In constructing more heavily trafficked roads, special measures are needed to avoid over compaction since this may make the soils weaker and more susceptible to the effect of moisture. In the extreme, it can produce a hard pan of clayey soil which interferes with natural drainage.

No special difficulties are experienced in measuring field densities, either before or after compaction. Field samples of these fine-textured soils can normally be obtained by the core cutter method. The samples of soil which have been oven dried to determine their dry weight should never be used for subsequent testing. Samples taken for other tests should be carefully handled, kept in sealed containers and not subject to high temperatures or agitation before testing. Particle size analysis by sieving will only be necessary when the soils are found to contain particles larger than silt size. Indeed, the agitation during sieve analysis is likely to cause an unquantifiable breakdown of the soil structure. For the sedimentation test, sodium hexametaphosphate is usually an effective dispersant but it may be necessary to use higher concentrations than normal to obtain full dispersion before carrying out the test.

Plasticity tests present a dilemma since the soil fines used in these tests must be fully worked before testing. Such working reduces these soils to a condition quite different from the state in which they occur in the field and unrepresentative of their engineering properties. With such soils, the chief value of the plasticity tests is as an indication of their susceptibility to change in texture as they are worked.

Compaction tests are affected by the same difficulty. A quantitative estimate of the effects of compaction on the soil structure can be obtained by comparing the results of sedimentation tests on samples of soil that have been subjected to different compactive efforts i.e. using the British Standard light and heavy compaction procedures. For the same reason, the standard laboratory CBR test may not give a reliable indication of the bearing capacity of these soils and it is best to derive design values for pavement design purposes from field CBR measurements on existing compacted soil subgrades. In virgin areas it will be necessary to carry out field trials to determine the most suitable ways of compacting these soils and to establish appropriate CBR values for pavement design. In settled areas, local experience will be available to help establish optimum methods and levels of soil compaction and appropriate CBR values for pavement design. Compaction with equipment that does not subject the soil to high and concentrated shear stresses will be preferred.

Some volcanic ashes may show little sign of weathering. In New Zealand in the upland plain south of Rotorua, volcanic ash from up to three successive recent eruptions can be seen exposed in road cuttings. In this temperate climate the grey ash shows little sign of weathering, except in the thin layers of organic soil which developed on the surface between eruptions. This material can be tested and compacted by normal means; it is unlikely that weathering will weaken it under roads in the foreseeable future. But allophanic and halloysitic clays do occur in New Zealand and Jacquet (1989) has reported on their loss of strength on remoulding. Where soils deriving from volcanic ashes occur in hot and humid climates, it will always be desirable to look for signs of chemical alteration and to proceed with engineering design and construction according to what is found.

3.3.2. Expansive clays

These are soils that are characterised by large volume changes with changes in moisture content. The term 'expansive' risks confusion with the over-consolidated clays that occur in northern latitudes. These latter are clays that have been compressed by the weight of ice during past glaciations and they are truly expansive since they undergo a permanent increase in volume when they are disturbed. The soils referred to here are distinguished by containing smectite, a group of clay minerals probably more familiar to engineers under the name montmorillonite. Almost all clays have the characteristic of swelling and shrinking with changes in moisture content. Montmorillonite possesses this property to a marked degree.

Soils containing montmorillonite are widely distributed in tropical and sub-tropical regions. These are the black cotton soils of Africa and Asia and the gumbo soils of central America. It is likely that they originally derived from the weathering of basic igneous rocks and that they developed in hot humid and alkaline conditions, possibly even under water. Montmorillonite is a persistent clay mineral. It can be found as a constituent of sedimentary shales and mudstones and it frequently occurs as a constituent of transported soils, usually mixed with other clays and silt as in the Nile delta.

Montmorillonite also occurs as concentrated clay deposits. The soil in such deposits undergoes dramatic changes in character between wet and dry seasons. During the dry season it shrinks rapidly, developing high cohesive forces which cause the clay to fissure in a typical block pattern (Plate 3.5). These fissures often penetrate for 5 metres or more. With the coming of the rains the soil re-expands rapidly. The cracks close and the soil becomes sticky and intractable. There is an area of black cotton soil near Selima on the western coast of Lake Nyasa where the level of the ground surface is said to change by some 25 centimetres between the wet and dry seasons. The soil in such deposits is very uniform in texture. Particles of soil fall down the fissures in the dry season so the soil is in continuous circulation. Such concentrations of montmorillonitic clay are useless for agriculture and are avoided as foundations for roads and other structures. More commonly, montmorillonite occurs with other clay minerals. The term black cotton soil carries overtones of fertility but this fertility does not derive from the montmorillonite. In the Nile delta for example, it derives from the fresh silty soil brought down by the annual flooding of the Nile.

The amount of montmorillonite present in a soil can be determined quantitatively by X-ray diffraction analysis, but such examination is expensive and tedious and is not necessary in normal survey and design. Soils containing less than 20 per cent by weight of clay are not likely to give trouble from shrinking and swelling, even where there are extremes of dry and wet in the annual climatic cycle. With more clayey soils, an indication of their swelling and shrinkage characteristics may be obtained using the shrinkage tests described in Chapter 2. Head (1984) has described the use of the consolidation test apparatus (Fig. 2.14) as a means of measuring the swelling characteristics of expansive clays. As described by Higgs (1988), methylene blue can be used to identify, and reject for civil engineering use, any shales or mudstones containing montmorillonite.

Where road building necessarily runs over montmorillonitic soils, there are two questions to be resolved. First, is the local climate such that changes in the moisture content of the soil can produce shrinkage and swelling that will damage roads and other structures built on the soil? If so, what engineering measures are possible to minimise or prevent such damage?

In temperate climates, seasonal moisture changes are usually not sufficient to merit special measures. In England, London clay, Gault clay and Oxford clay all contain montmorillonite but it is only during the rare very dry summers that shrinkage causes damage to roads and buildings. On roads, this damage is usually minor and the costs of repair are much less than those which would arise from taking precautions in design. There is a current trend to take precautions in building new houses and other structures on these soils. In Montmorillon, a town in central

France, care has been taken with structural foundations ever since these soils were identified and named after the town. Some of the local roads bear the tell-tale mark of longitudinal cracks, now effectively sealed, about a metre from the kerbside. Such cracks are symptomatic of roads over expansive soils.

In warmer climates with pronounced dry and wet seasons, special engineering measures are necessary to prevent damage to roads and buildings. Chen (1988) gives comprehensive information on the derivation of expansive clays and on the design and construction of foundations over them. Areas of concentrated montmorillonitic clay which exhibit the intense seasonal fissuring are best avoided altogether but the expansive characteristic of these soils is usually abated by the presence of other constituents in the soil. The engineering measures to be used with these soils are reviewed in Chapter 11. Here it is sufficient to say that these measures have three essential features.

(a) Seeking, where possible, to avoid such soils by locating roads elsewhere. One of the first applications of aerial photographs for this purpose was in Northern Nigeria in tracing road lines on relict raised beaches around Lake Tchad.

(b) Where construction over these soils is inevitable, seeking to construct embankments and soil subgrades at a moisture content close to the equilibrium value that will be reached under an extensive sealed surface. As Ruckman (1980) has reported, in dry climates this may involve wetting the soil before construction. Strongman (1963) has described a most successful application of this technique in Kenya where a bridge embankment on the Nairobi-Mombasa road, some 8 metres high, was constructed with montmorillonitic clay, compacting the soil to a uniform density as it dried out at the end of the wet season. The sloping faces of the embankment were covered with rip-rap to minimise moisture changes from rainfall and evaporation. Measurements in the succeeding years showed negligible swelling or shrinkage and no sign of the fissuring so common in the surface of roads built over such soils.

(c) Taking particular care that the roads are designed and maintained so that surface water cannot penetrate to the soil subgrade during the rainy season, either through the surface or from the road sides.

Land-system classification (Chapter 2) can be helpful in defining those parts of a region where such soils are likely to occur and the indications of their presence on the ground. There are no special precautions necessary in undertaking classification or engineering tests on these soils except that those who insist on undertaking CBR tests on specimens that have been soaked in water for four days are likely to find that the soils have started to swell out of the test moulds.

3.4 Desert soils

Deserts are inhospitable places, usually uninhabited except, perhaps, by nomadic people. Since early times, traders have beaten out trails across them, often for immense distances. For example, the silk route between China and the Levant traverses desert lands over much of its route, and

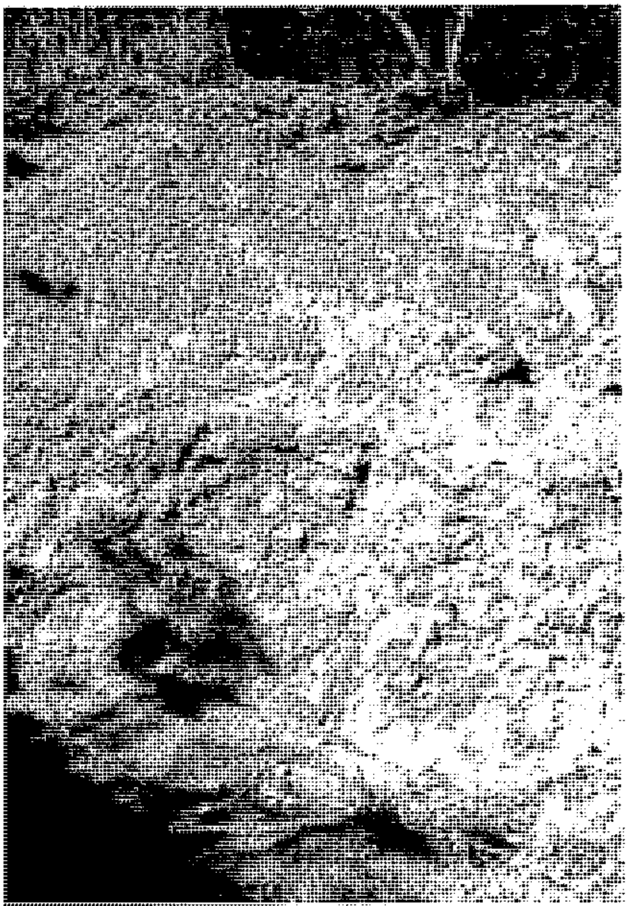
Plate 3.1 Nodular gravel.


Plate 3.2 Calcrete pan.

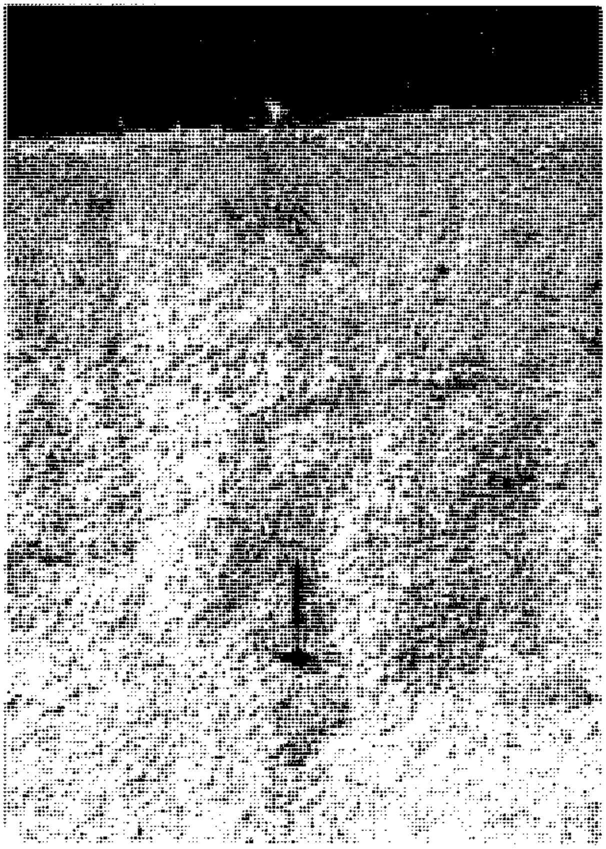

Plate 3.3 A transported sandstone gravel used in the base of the airport runway in Brunei.

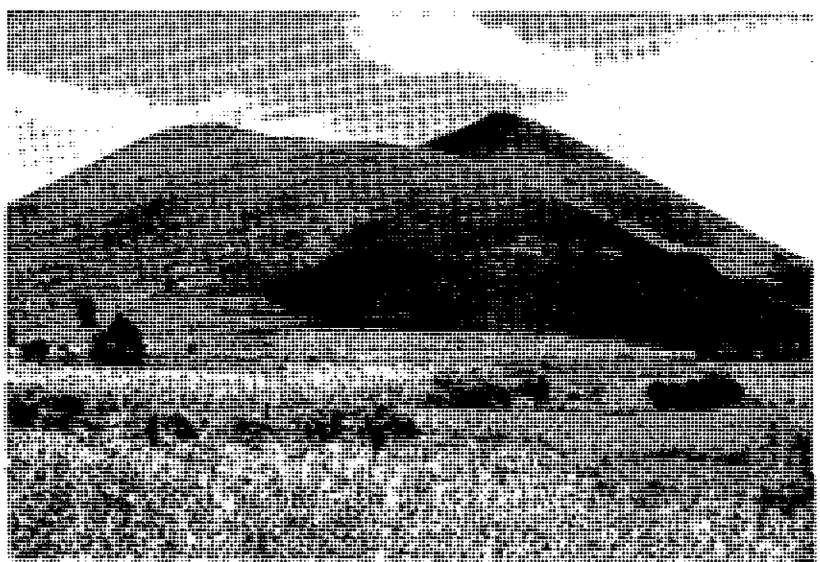

Plate 3.4 Volcanic scoria in Ethiopia.

Plate 3.5 Typical block shrinkage pattern of montmorillonite (black cotton) clay soil.

Plate 3.6 Sinuous sand track through the Kalahari desert.

caravans travelled for many years across the southern Sahara between Senegal and Egypt. This immense effort was justified by the great value of the cargoes which were carried, and the travel costs were low, the traders living off the land in the manner of the nomads. Indeed, desert travel was an advantage in that the tolls that would be exacted from them in traversing more populous lands were avoided.

All this has changed. Precious goods can be transported by air, there is oil in the desert and there are other commodities, such as cattle and crops, which are raised in fertile areas enclosed by desert lands. Roads are needed, both to realise the potential of this wealth and to distribute the goods that can be purchased with it. Vehicles with four-wheel drive and low pressure tyres can go some way in simulating the mobility of camels over desert sands but they are expensive to operate and passage is often uncertain. Roads with hard surfaces are needed and it is important to know how to employ desert soils to make them.

The predominant surface soils in most deserts are wind-blown sands. We can start by dispelling a popular misconception about sandstorms. The dust which darkens the sky in such storms is not sand. It consists of fine silt and clay particles. The sand in such storms never rises more than 2 metres above the ground surface. It bounces along in low trajectories close to the ground, moving dunes in a down wind direction and piling up around obstacles in its path. It is characteristic of these storms that the dust clouds clear the immediate area within a few hours whilst the sand continues to move over the surface of the ground long after the sky has cleared. The dust may travel for many miles before it settles. This is the origin of the loess soils found in Northern America, Northern Europe and Asia. Even today, dust raised by the Harmattan wind in the Sahel region regularly darkens the skies of coastal areas of West Africa and occasionally travels as far as Northern Europe before it is brought to the ground in rain. The sand, in the meantime, has travelled only a short distance, settling to reshape the surface of the ground. The sand almost invariably consists of rounded or sub-angular particles of quartz and one effect of the wind is to sort it locally into deposits of different uniform size (see Bagnold (1984)).

In deserts with moving sand, a primary concern of the road engineer is to locate and maintain the road so that the surface is kept clear of sand. The ground surface along the road line, and over the road itself, should be kept as smooth as possible, perhaps even raising the pavement slightly above the surrounding ground to increase the velocity of the wind so that blown sand is carried clear of the road. Roads should be located on the windward side of obstacles; the direction of prevailing winds rarely varies by more than ninety degrees. Side drains are usually not necessary and the interruption they make in the smoothness of the ground profile should be avoided whenever possible. Fences, such as those used to control drifting snow, are sometimes suggested but a fence will usually initiate a dune which will soon engulf the road. Where there is a risk of occasional flash flooding from nearby mountains, roads should be located some distance away from detritus fans i.e. at least half as far again from the foot of the fans as they extend from the mountain slope.

When settled in place, desert sands are often sufficiently strong to carry the occasional vehicle, and drivers soon learn to avoid local areas of recently accumulated wind-blown sand. Vehicles, following the same track, produce ruts which may become sinuous as the vehicles bounce from side-to-side, as shown in Plate 3.6. The first stage in improving such roads may be to surface them with gravels, provided there are local deposits of such material available. But gravel roads are rarely very durable and it is difficult to provide the regular maintenance they need. On roads carrying regular traffic of more than 50 vehicles per day, there is a well justified demand for a bituminous-surfaced road.

In Zimbabwe during the 1940's, strip roads were built over the Kalahari sand. These were single-lane roads paved only on two strips in the wheel tracks, each strip being about 0.8 meters wide and made from broken stone with a bitumen grouted surface and with hard gravel shoulders. In the days of hand construction, this produced a saving in construction costs but it was impossible to maintain the edges of the bituminous strips. In passing or overtaking other vehicles, one soon learned to take to the gravel shoulder so that it was the other driver's windscreen which was broken by flying stones.

There are two particular problems in providing bituminous pavements over desert sands. One is that the sands usually occur at low field densities and therefore compaction to a depth of one metre may be necessary to prevent uneven settlement under the loads of heavy vehicles. The other lies in finding suitable materials to construct the pavements. During survey, density profiles of the sand should be undertaken to a depth of at least one metre below the level of the intended finished road surface. Measurements of the state of compaction under the existing road or under adjacent roads carrying heavy vehicles can indicate the amount and depth of the compaction needed. Compaction always raises problems in areas where water is a precious commodity. Unfortunately, the methods of dry compaction developed for use in arid areas are not very useful with these cohesionless and often quite single-sized sands. One method of economising on water for compaction, which had been used with some success in Southern Africa (Todres 1970)), lies in the use of surface-active agents dispersed in the water. By lowering the surface tension of the water its wetting powers are increased, improving the lubrication between the sand particles and thus lowering the optimum moisture content for compaction. In both Australia and South Africa the use of heavy impact rollers has been found effective with desert sands. Claims are made that they can produce effective compaction to depths of up to 4 metres (Clifford (1980), and Van Rooyen and Wessels (1967)).

Collapsing sands occur in some areas, most commonly with wind-blown sands which have acquired a fragile structure, as they were laid down, from the presence of clay particles washed into them from rain falling through dust clouds. This clay accumulates around the points of contact of the sand particles, exerting suction forces as the soil dries out so that the soil acquires a structure sufficiently strong to resist compaction under the weight of further deposits of sand. These soils have low field densities and can collapse dramatically, when loaded, to occupy less than 80 per cent of their original volume.

This phenomenon is of particular concern with heavy engineering structures such as dams and buildings. It first came into prominence in Africa after the sudden settlement by one metre of a mine heading under construction in northern Zambia. Jennings and Knight (1975) have described these soils and the remedies available in using them as foundations. Where such soils are found during ground exploration, it may be possible to induce collapse by a combination of flooding and vibratory or impact compaction on the surface. Otherwise it is necessary either to remove the collapsing sand, replacing it with more stable material, or to pile through the collapsing sand to firm strata below. No dramatic examples of the collapse of road pavements from this cause are recorded but it could presumably occur, particularly under heavy embankments. In South Africa there is an insistence on full compaction of such soils for at least one metre below the formation level of road subgrades. No doubt this is a necessary precaution in building heavily trafficked roads, even deeper compaction being needed on airfields. But on more lightly trafficked roads in desert areas, neither deep compaction nor replacement of the soil are very practicable.

Residual soils developed from granitic rocks may display the same phenomenon. Indeed, such collapse is a common initiator of landslides on steep slopes where a build up of moisture in the soil at the foot of the slope induces collapse. Lumb (1962) has described the properties and behaviour of such soils in Hong Kong. In building roads or other structures through such terrain, it is always desirable to make sure that the works improve rather than impede the natural drainage pattern.

Deposits of concretionary gravel may be available locally for the construction of the roadbase in desert areas. More rarely it may be possible to find local outcrops of rock suitable for quarrying. Human habitation and a source of water are often found near such outcrops and they provide an obvious location for the siting of quarries and road construction and maintenance depots.

Desert sands are not usually suitable for stabilising with either cement or lime but they can often be effectively stabilised with bitumen. It may be necessary to combine sands from two or more sources to obtain a stable grading and therefore, where there is the prospect of bitumen stabilisation, the field survey should include exploration to establish the presence of deposits of sands of suitable grading along the line of the road. A good example of the effective use of bitumen stabilisation is on the Maiduguri-Bama road in Northern Nigeria, reported by Johnston and Gandy (1964). Here, a pavement of sand stabilised with bitumen, 125 millimetres thick, was still giving good service twenty years after construction. More detailed information on bitumen stabilisation is given in Chapter 6.

No special difficulties occur in classification or engineering tests on these soils. Vibratory methods are most suitable for compacting them and the vibratory method of compaction should be used in standard compaction tests. The bearing capacity of such sands is much affected by the extent to which they are confined. Close confinement enables them to muster the internal friction from which their stability derives. In undertaking CBR tests, surcharge weights are therefore used to confine the compacted soils in the test moulds.

Soluble salts covering quite large areas of low ground often occur in some deserts, notably in the Northern Sahara and in the Arabian peninsular. These salts are usually the chlorides and sulphates of calcium, magnesium and sodium. The deposits are best avoided by the road builder but in areas near the sea this may be difficult. The presence of small quantities of these salts does no harm in soil embankments nor in stone bases but it can produce rapid corrosion of metals used in reinforced concrete and in drainage installations. Salt-bearing sands should be washed free of salt before use in making concrete for steel reinforced structures, and the concrete should be well-made, dense and of low permeability, with care being taken to provide adequate cover over the reinforcement. In extreme cases, cathodic protection of reinforcement will be necessary.

These salts have no deleterious chemical effect on bituminous materials, but blistering of thin bituminous surfacings can occur where they are present in the soil below the road or in the road making materials. Such blistering occurs in hot arid or semi-arid areas where evaporation exceeds precipitation, bringing saline ground water upwards to the ground surface. These salts are hygroscopic to some degree, so that a daily cycle develops, water evaporating through the surfacing during the heat of the day, with more salt-laden water being brought in from below during the cool of the night. Obika *et al* (1989) have reviewed reported experience. They quote suggested limits for the salt content of aggregates used in road building. A dense bituminous surfacing, at least 30 millimetres thick, inhibits the day-time evaporation and reduces the rate at which the salt crystals develop. Over extremely saline soils it may be necessary to use either a coarse-grained layer of aggregate or some form of geofabric at sub-base level to inhibit the upward movement of salt-laden water.

Some saline soils also contain carbonates amongst the deposited minerals, for example, the sabkhas in coastal areas in the Arabian peninsular and similar deposits in Namibia and in other desert areas. These soils provide an excellent running surface in dry weather, the carbonates providing mechanical stability and the deliquescent chlorides helping to maintain them free from dust. They vary a great deal in composition and, when rain falls, ruts quickly develop in weak places. Ellis (1973) has reviewed experience in using this material for road foundations and discussed the prospects for using material from areas of high carbonate content as road bases.

Desert soils are almost invariably sandy or rocky. There is an exception in the Australian outback, notably in Queensland. This soil is locally known as 'bull dust'. The particles are of silt and clay size with the predominant clay mineral said to be montmorillonite. But its behaviour is very different from montmorillonitic clays elsewhere. On the rare occasions when rain falls, its sticky clay nature rapidly becomes evident but, as it dries out, there is only a small range of moisture content within which it exhibits any cohesion. As it dries further under the intense dry heat, it slakes to a fine powder, an all-penetrating dust which arises in clouds to mark the passage of vehicles and herds of cattle. It seems likely that if high winds were prevalent in the Australian desert, this soil would long ago have been blown away, perhaps to form loess soils elsewhere. Meanwhile it remains one of the trials of life in the Australian outback.

3.5 Miscellaneous soils

Mica is a constituent of many granites and other acidic rocks and may be present in residual soils derived from these rocks and from gneisses. Its presence may easily be detected visually during survey, and once more the land-form classification system can be used to identify those particular areas in a country where micaceous soils are likely to occur. The presence of mica makes these soils particularly susceptible to erosion by water and, when present in quantity, the mica produces difficulties in compaction. It imparts marked elastic properties to the soil so that the soil rebounds after the passage of compaction plant. There are, as yet, no reliable and simple testing procedures for measuring the mica content of soils nor of prescribing limits of mica content beyond which these soils are unsuitable as road foundations. Compaction equipment which imparts high and concentrated shear stresses to the soil e.g. sheepfoot rollers, are likely to be most useful in compacting these soils to a stable condition. It is best to plan the route of the road to avoid highly micaceous soils. They are often encountered in cuttings, erosion by water having removed the soils from adjacent valleys. In these situations it may be necessary to remove the soil in the cuttings to a depth below the invert of the side drains, replacing it with non-micaceous soils brought in from elsewhere.

Unconsolidated silty clays and clayey silts occur in deltaic areas and in the coastal fringes of many tropical countries. These are similar in nature to the estuarine soils found in more temperate regions except that, because many of the world's great rivers discharge into tropical seas, the deposits are therefore deeper and more extensive. Such soils are usually very fertile, as in the Nile delta, the rice bowl of Thailand and in Bangladesh. They support dense agricultural development and, although much of the transport can be done on water, there is an increasing demand for road transport which is immune from the frequent flooding in these areas. It is sometimes possible to locate roads on sand-bars which occur near the shore line of some coastal deposits. Design and construction techniques over these soils are reviewed in Chapter 11.

Organic soils are rare in the tropics in areas that are not perpetually flooded since the products of rotting vegetation are soon leached out of the soil. Some soils may occasionally be found where the remnants of this decay interfere with their stabilisation with cement, and, when soil-cement mixtures do not harden as they should, this is one possible cause to be investigated. Wetlands are not uncommon in the tropics, for example in the Sud of the southern Sudan, in the papyrus swamps of Uganda and in many flat coastal areas. These areas do not usually support large populations and the road engineer is likely to encounter them only in providing links through them to connect areas of human activity on either side. Here, an aerial survey can be of value in locating more supportive soils, for example, the sand-bars which occur parallel to the coast in the swampy coastal areas of Guyana, as described by Bryan (1958). In shallow and stagnant swamps it may be possible to consolidate the soils by the same methods that are used with unconsolidated soils, as was done in building the road over the Caroni swamp in Trinidad, reported by Osborne (1960). Attempts have been made to float roads over such swamps since prehistoric times; witness the timber-cord tracks found preserved in such swamps in many parts of the world. Construction of pavements supported on fascines may be a useful temporary expedient for roads carrying light motor traffic. Hovercraft are a rather expensive possibility, an expense which may be justified by the value of the commodities carried. In the limit, it may be necessary to bridge such areas, founding the bridges on piles penetrating to firmer strata below the swamps. The expense of such bridging may be justified for particular reasons, for example, in the urban motorways traversing the densely populated areas of Lagos in Nigeria.

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4 Rocks

4.1 Introduction

The rocks which make up the earth's mantle are of three distinct modes of formation, igneous from the hardening of molten rocks, sedimentary from the deposition, usually under the sea, of the weathered products from igneous rocks, and metamorphic in which rock from either of the two foregoing groups has been altered under pressure and heat as a consequence of the tectonic movements and volcanic action in the earths crust that are still taking place, though much less violently than in the geological past. Within each group there are many forms which have been classed in specific geological subgroups but it should always be remembered that rocks change by insensible gradations between adjacent groups and therefore there are likely to be variations in mineral content and in quality within any deposit. The best quarry sites are usually those where the rock is monolithic and massive with only small variations in nature and quality.

Most rocks can be used in road making. The nature and qualities of the first two groups are summarised in Table 4.1 (igneous) and Table 4.2 (sedimentary).

Metamorphic rocks can be formed from almost all igneous and sedimentary rocks. Metamorphosis by contact with intrusive molten rock can change their mineralogical structure so that the nature of the original rock is completely obliterated. The resultant rock, hornfels, usually shows interlocking of the mineral crystals. It is fine grained and usually very strong and unfissured, and it can be quarried to make an excellent road stone. Stone that has been more weakly altered by heat can often be found amongst granite and basalt deposits. Quartzite and marble are weakly metamorphosed forms of sandstone and limestone respectively.

Rocks metamorphosed by pressure alone usually possess a defined laminar structure. In gneiss and granulite, the banding is well separated and the rocks resemble coarse-grained granites and have similar properties. Slates and schists are metamorphosed from shales and mudstones and the laminations are more closely spaced. They may be of value in building but find no use in road making.

4.2 The weathering of rocks

In the tropics it is commonly found that rocks exposed on the surface are deeply weathered. In this respect they differ from the exposed rocks in temperate regions. Weathering proceeds quite slowly in cooler climates and often the mantle of weak weathered rock has been removed by successive glaciations so that it is rare to find a surface rock that is weathered deeply. Things are quite different in the tropics. Here, high temperatures, often associated with high humidity, can produce physical and chemical changes to a considerable depth in surface rocks. In dry areas the weathering is predominantly physical. The rock is shattered by alternate heating and cooling and

TABLE 4.1
IGNEOUS ROCKS

ACID (over 66% total silica)	INTERMEDIATE (52-66% total silica)	BASIC (less than 52% total silica)
COARSE GRAINED (PLUTONIC) Grain size larger than 1.25mm. Generally suitable as roadstones but inclined to be brittle. Very large grain size makes for attractive masonry but diminishes the value of the rock as roadstone		
GRANITE GRANODIORITE	SYENITE DIORITE	GABBRO NORITE
MEDIUM GRAINED (HYPABYSSAL) Grain size between 1.25 and 0.1mm, often with interlocking crystals. Usually makes good roadstone		
MICROGRANITE GRANOPHYRE	PORPHYRY PORPHYRITE	DOLERITE DIABASE
FINE GRAINED (VOLCANIC) Grain size below 0.1mm and hardly distinguishable to the naked eye. Usually makes good roadstone but can be brittle and splintery. Some can become rather polished under traffic		
RHYOLITE FELSITE	ANDESITE TRACHYTE	BASALT SPILITE

Light colour ____________________ Dark colour
Low Specific Gravity (2.6) _______ High Specific Gravity (2.9)
Increasing affinity with bitumen ——>

The minerals in surface deposits of igneous rocks in the tropics are always likely to be weathered. The depth to which they are weathered varies, being greatest in humid climates and in rocks that are shattered by tectonic folding. Such weathering may only be slight, e.g. as in the brown staining often evident in spots on granite masonry. It is likely to be greatest with more basic rock, and less evident from hand inspection. In exploring for quarry sites it is always necessary to establish the depth of such weathering, with the intention of quarrying only the rock which is unweathered.

the broken rock may be quite similar in nature to the rock from which it comes. In more humid areas, chemical weathering proceeds quite rapidly and many of the rock minerals will be found to be at least partially weathered towards their ultimate clayey form.

Such deterioration may not be obvious in a hand sample and the consequences of using such weathered rock in roads may be disastrous. An early example in Southern Africa provided a warning of these dangers. The terrain through which the road was built was littered with rounded boulders of dolerite. These, crushed down to an appropriate size, had been used for many years to provide a broken stone running surface. When the road was improved, this same material was used to make a crushed stone base under a bituminous surfacing. It served well for several years

TABLE 4.2
SEDIMENTARY ROCKS

CALCAREOUS
Predominant mineral: calcite $CaCO_3$

LIMESTONE Softer than igneous rocks but usually of adequate strength for most road making purposes. Usually light in colour. Good affinity for bitumen. Specific Gravity 2.65-2.75
Proclivity to polish under traffic so avoided for use in surfacings on busy roads in wet climates

DOLOMITE Limestone in which part of the calcite is replaced by dolomite ($CaMg.(CO_3)_2$). Similar to limestones but slightly stronger

CHALK, WEAK LIMESTONES, CORAL
Useful for embankment construction. Can occur in places where no other rocks are locally available and can then be used for bases and surfacings. See the notes on these materials given in this Chapter.

SILICEOUS
Predominant minerals: quartz and chalcedony, both SiO_2

SANDSTONE Usually stratified. Weak when of recent geological origin. If older than Carboniferous may be hard enough for use as roadstone. Variable affinity for bitumen. Specific Gravity 2.6-2.75

QUARTZITE Very hard and usually of adequate strength for roadstone but may be rather brittle. May have high resistance to polishing under traffic. Variable affinity for bitumen. Specific Gravity 2.55-2.65.

FLINT Derives from the deposition of silica transported in solution from elsewhere. As with quartzite, frequently occurs as a water worn gravel.

ARGILLACEOUS
(Clay minerals predominate)

SHALE Soft fine grained and usually laminated. Can be used in making embankments but unsuitable for other uses in road making

MUDSTONE A harder and older variety of shale which may be hard enough for use as a roadstone.
The clay minerals from which these rocks are formed may include montmorillonite. Wben montmorillonite is present the rocks are likely to be unstable when exposed.

then, quite suddenly, ruts began to appear in the surface. When the road pavement was opened up, there was little sign that rock had ever been used in the base. It had broken down to become a sticky yellow clay. An inspection of the remaining boulders showed brown discolouration on the surface but the rock appeared to be sound inside the boulders. The boulders had a convincing ring when hit with a geological hammer and, under hand inspection, the crystals of the different rock minerals appeared to be intact. Subsequent mineralogical examination showed evidence of chemical decomposition in these minerals and it seems that this chemical breakdown had been greatly accelerated under the traffic stresses and steamy heat in the roadbase. This occurred over forty years ago. Since then, similar failures of road and airfield pavements in different parts of the world have been reported and there is a general awareness that hand inspection will not necessarily reveal fairly advanced chemical deterioration of rock minerals.

There are three possible approaches in preventing the quarrying and use of such weathered rock. The first is confined to areas of dry climate, taking advantage of the fact that the weathering of rocks in hot dry climates is predominantly physical. In an extensive survey in Southern Africa, Weinert (1974) discovered that it was only the rocks occurring on the wet side of the 500 millimetre isohyet that were affected. It is a happy coincidence in this region that boundaries in the present climate remain indicative of events in the geological past. It considerably simplifies the work of surveying for possible quarry sites to know that chemical weathering of rocks is not likely to be a danger in parts of that region where the average annual rainfall is less than 500 millimetres.

Elsewhere in the tropics it is always desirable to undertake mineralogical examination of samples taken from rock drilling carried out during exploration for quarry sites. All igneous and some metamorphic rocks must be suspect. Sometimes there will be guidance available from local geological institutions on the need for such testing. When in doubt, mineralogical examination should be done, and sites where there are obvious indications of mineral decomposition in the rocks should either be avoided or the rock overburden should be removed to a depth below which there are no signs of deterioration from chemical weathering.

The third method involves the use of a land-form classification system as outlined in Chapter 2. With such a system available in a country, it is easy to collect and store information on the surface rocks in each land-form, on their likely location and mode of occurrence, and on the extent to which each rock is subject to mineral decomposition. By this means, it will be possible to eliminate some rock formations from suspicion and to know the extent of mineralogical testing needed when seeking quarry sites in suspect formations.

There are relatively simple laboratory tests which can be used to indicate whether samples of rock chippings contain particles of suspect hardness and durability. The best known of these is the magnesium sulphate test originally developed for examining the stability of concrete aggregates. In this test, a sample of dried aggregate chippings of specific size is immersed for a period in a saturated solution of magnesium sulphate. The sample is then oven dried at 105-110°C. This immersion and drying is repeated for a specified number of cycles. The formation of sulphate crystals within the chippings disrupts the weaker chippings and the result is recorded as the proportional reduction in chippings of the specified size (see Minty and Monk (1966) and the relevant National Standards).

This test will certainly identify rocks which are weak and porous. Its use to identify rocks that are partially weathered is less certain since some of these are not likely to be very porous and the test is only a very crude simulation of the continued weathering which will afflict the crushed stone in a roadbase over a period of years. It would be foolish to use it in a specification as the sole means of establishing the susceptibility of rocks to weathering over a period of years in an engineering structure. In evaluating igneous or metamorphic rock for possible quarry sites, it is always advisable to have samples of the rock subject to expert mineralogical investigation.

The right course to follow, when offered crushed rock by a contractor, is first to establish what local experience is available in its use and in the use of rocks of the same mineralogical type in the vicinity. The next step is to inspect the quarry to see if there is any sign of chemical weathering e.g. brown staining on exposed vertical faces. Such enquiries may be sufficient to give the rock a clean bill of health. If grounds for suspicions remain, then magnesium sulphate tests could be undertaken, preferably on chippings made from rock taken from the quarry face adjacent to any fissures. It will always be wise to seek advice from Geological Departments in Government and Universities. They will usually be pleased to take such opportunities to enlarge their knowledge of local geology.

One factor influencing the use of rocks, particularly in making concrete, is their coefficients of thermal expansion. There are considerable differences between different rocks and these differences are reflected in concrete made with them. Typical figures are given in Table 4.3.

TABLE 4.3
EFFECT OF AGGREGATE ON THERMAL EXPANSION OF CONCRETE

Aggregate used in concrete	Coefficient of thermal expansion of concrete per °C x 10^{-6}	
	Range	*Average*
Quartzite	11.7 - 14.6	13.2
Flint gravel	9.0 - 13.7	11.4
Granite	8.1 - 10.3	9.2
Basalt	7.9 - 10.4	9.2
Limestone	4.3 - 10.3*	7.3

* a siliceous limestone

4.3 Corals and soft limestones

These occur widely in tropical areas. Much of the land mass is surrounded by coral reefs and there are the coral atolls in the Pacific and Indian Oceans. In the Caribbean and around the Bay of Mexico, many of the islands and parts of the land mass consist of limestones of varying hardness raised from the sea bed by tectonic movements. Some of these limestones are quite hard and can be treated as ordinary limestone aggregates for road making purposes. Others, including coral, are soft and require special treatment in road making.

On some coral islands, a volcanic cone obtrudes which can provide rock for roads and building. Others consist entirely of coral and their ground surface is rarely more than two metres or so above sea level. Apart from the current alarm about global warming and a rise in sea levels, there are good ecological reasons for taking care in selecting coral for use in road building. 'Green' coral i.e. coral in the surface of the living reef, should never be used. Extraction of such coral could do irreparable damage. Dead coral is easily excavated for use in roads and buildings and, unless done to excess, there can be no ecological objection to this. Indeed, roads on a raised bund along the shore line can provide a defence at places vulnerable to erosion by the sea.

Even though the strength of these corals and soft limestones falls short of the normal requirements for roadstones, they are excellent materials for embankment construction, and deposits of harder material can be selected for use in making roadbases, bituminous surfacings and in the manufacture of concrete. Beaven (1971) has prepared a review of the properties and uses of corals and soft limestones in road building. Table 4.4, taken from this review, shows the range in engineering properties found in the marly limestones on Caribbean islands. Similar material, known as caliche, has been used for many years in the southern states of the USA, notably in Texas where it is frequently the only available source of aggregate. It can be used as dug for embankment construction, grid-rollers and sheeps-foot rollers being the most suitable compaction equipment. Subgrades of soft limestone and coral normally have a high bearing capacity, their CBR values for pavement design being above 15 per cent. For use in bases, surfacings, and concrete, it is necessary to seek out deposits of the hardest material locally available. Hardness is usually reflected in the density of the material and a convenient method of indicating hardness is to determine the specific gravity and water absorption of samples.

Quite effective bituminous surfacings can be made with the harder limestones and corals. For airfields, it may be necessary to design asphalt mixtures using the full rigours of the Marshall testing procedure (see Chapter 6), but for roads, it is simpler and probably more satisfactory to derive specifications from local experience, defining them by the grading of the aggregate and

TABLE 4.4
PHYSICAL PROPERTIES OF SOME WEST INDIAN LIMESTONES

	Source	*Ten per cent fines value Mg*	*Aggregate impact value*	*Aggregate abrasion value*	*Polished stone coefficient*	*Water absorption (per cent)*	*Specific gravity*
Jamaica	Enfield quarry* (St Mary)	19.5	19	7.9	0.71	1.5	2.61
Jamaica	Yallas quarry (St Thomas)	10	29	17.3	0.53	4.4	2.32
Trinidad	Central range	10	24	34.0	0.49	3.4	2.40
Trinidad	Eastern quarry (North range)	8.5	29	18.0	0.33	1.4	2.61
Barbados	Husbands quarry	8.5	34	18.3	0.56	3.0	2.38
Barbados	Penny Hole quarry	7	35	19.9	0.53	4.4	2.27
Bahamas	Eleuthera rock	7	30	28.4	0.63	7.3	2.20
Bahamas	Eleuthera chippings	7	34	49.5	0.61	12.8	1.93
Bahamas	Anglo Colonial	5.5	38	47.0	0.63	7.2	2.09
Bermuda	Harrington Sound quarry						
	Crushed aggregate	5.5	42	31.8	0.57	4.5	2.30
	Poor lump stone	3	46	43.5	0.62	3.7	2.35

*contains siliceous material

TABLE 4.5
SPECIFICATIONS FOR BITUMINOUS SURFACINGS USING SOFT LIMESTONE IN BERMUDA AND JAMAICA

Grading Sieve sizes (mm) (BS 812)	*Bermuda Percentage passing (by mass)*	*Jamaica Percentage passing (by mass)*
20	98	100
14	77	95
10	65	85
6.3	53	70
3.35	43	47
0.6	28	20
0.212	10	10
0.075	5	7
Type of bitumen	RC 250	80-100 pen
Amount of bitumen (percentage of total mix by weight)	5.0	6.0

the type and amount of bitumen to be used in the mixture. Two typical specifications of this type are given in Table 4.5.

As with most limestones, such surfacings may become slippery when wet, but some contain silica particles and the effect of this in imparting quite a high polishing resistance (PSV) can be seen in the test results on the marly limestone from Jamaica quoted in Table 4.4.

Coral aggregates and sand have been used successfully for making concrete of good quality and durability. Corrosion of reinforcement might be a possible weakness, particularly when sea water is used in mixing, but experience suggests that this is a danger only when there are faults in design, in placement, and in the porosity of the concrete.

The high strengths which often develop in compacted corals and marly limestones has prompted a belief that these materials can have self cementing properties deriving from the mobilisation and redeposition of the calcium carbonate. Corroborative evidence comes from Malta where many of the buildings are made with masonry blocks cut from the soft local limestone. It is said that, as freshly quarried blocks dry out, carbonate is transported to the surface, giving the blocks a hard protective skin which can be lost if the faces of the buildings are redressed at any time.

Should such hardening occur in road foundations, it is an added unquantifiable bonus, but it finds no place in design procedures at present.

4.4 Testing of aggregates: intrinsic properties

4.4.1 Sampling

The properties of rock which are important for its use as roadstone include its resistance to crushing and abrasion, its specific gravity, its water absorption, its propensity to polish, and the size and shape of the crushed rock chippings. Size and shape of chippings are the result of the crushing and screening operations in the quarry, and regular testing is needed to make sure that these are consistently within specification limits. The strength, specific gravity and propensity to polish derive from the intrinsic nature of the rock, and tests are needed less frequently, first in evaluating the rock from a new quarry, and again later, as fresh faces of rock in the quarry are exposed for use.

The tests are all done on rock chippings of specific sizes. In evaluating rock from a prospective quarry, it will be necessary to prepare such chippings from hand-selected boulders using a small crusher. From an established quarry, chippings are taken from current production and great care is necessary to obtain representative samples. The relevant testing specifications give information on how this sampling should be done. Essentially it consists of taking incremental samples from the production line to build up a bulk sample of about 30 kilograms and thoroughly mixing this bulk sample before removing quantities of suitable size for testing. A riffle box (Fig. 4.1) is a convenient way of producing representative test samples from the bulk sample and the various National Standards for aggregate testing describe in detail how the sampling should be done. Whilst some rocks are quite uniform in quality and texture, others can vary considerably from point to point in a quarry face, therefore it is always important to follow the standard sampling procedures. In control testing during construction, the parties to the contract should always agree on the procedures to be followed for obtaining test samples. The grading of aggregates is likely to be changed during compaction and under subsequent traffic. Any specification for particle size analysis should therefore apply to material as delivered to the site and before it is laid on the road.

The relevant British Standard is BS 812, 'Testing aggregates', which is issued in 24 parts, each dealing with a specific test.

The various American Standards for testing rocks are described in the Annual book of the American Standards for Testing and Materials (ASTM) volume 04.02. The reference numbers for the relevant tests within this volume are quoted in the descriptions of the individual tests which follow.

4.4.2 *Aggregate Crushing Test and Ten Per Cent Fines Test* BS 812 (1990) Parts 110 and 111.

These tests are used both in surveys, to evaluate the crushing strength of available supplies of rock, and in construction, to make sure that minimum specified values are maintained. They require the use of a compression testing machine of at least 100 tonnes capacity and it is important that the testing machine be regularly calibrated for consistent results. There can be quite wide variations in the strength of rock in a quarry. For site control purposes, samples of the chippings

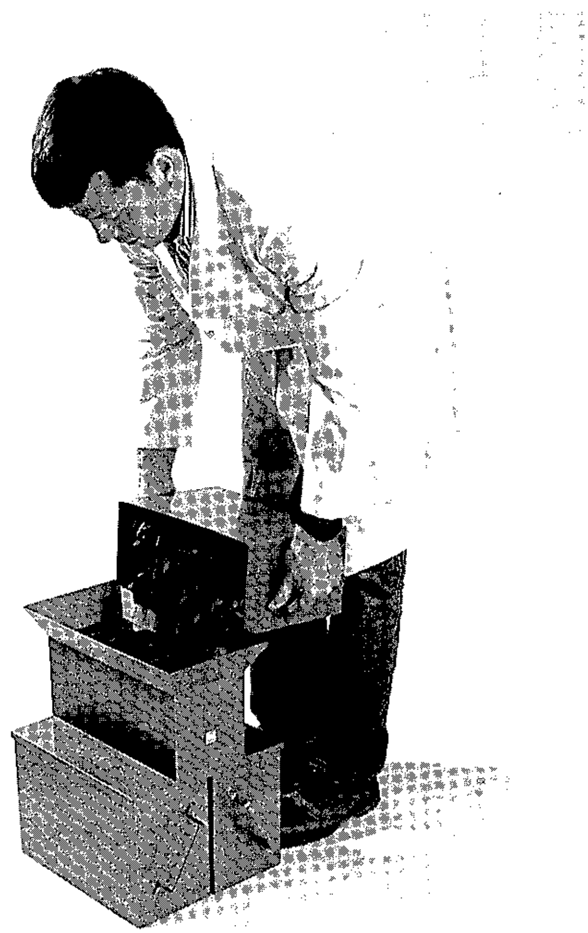


Fig.4.1 A riffle box for sampling aggregate.

should be regularly tested, at least once a month, until testing has indicated the range of strengths of the stone in the quarry. The test is undertaken using a metal plunger to apply a load to a sample of the rock chippings contained in a test mould as shown in Fig. 4.2. The test load is 40 tonnes in the Aggregate Crushing Test and the Aggregate Crushing Value (ACV) is the percentage by weight of fines passing the 2.36 millimetre sieve under this loading. Over the range of normal road making aggregates, ACVs vary from 5 per cent for hard aggregates to 30 per cent for weaker aggregates. With weaker rocks, the loading produces so many fines that they influence the effect of the loading and reduce the sensitivity of the test. For such weak rocks the same apparatus is used to evaluate the Ten Per Cent Fines value i.e. the load in tonnes which produces 10 per cent of fines passing the 2.36 millimetre sieve. This value is obtained by interpolation of the percentage of fines produced over a range of test loads.

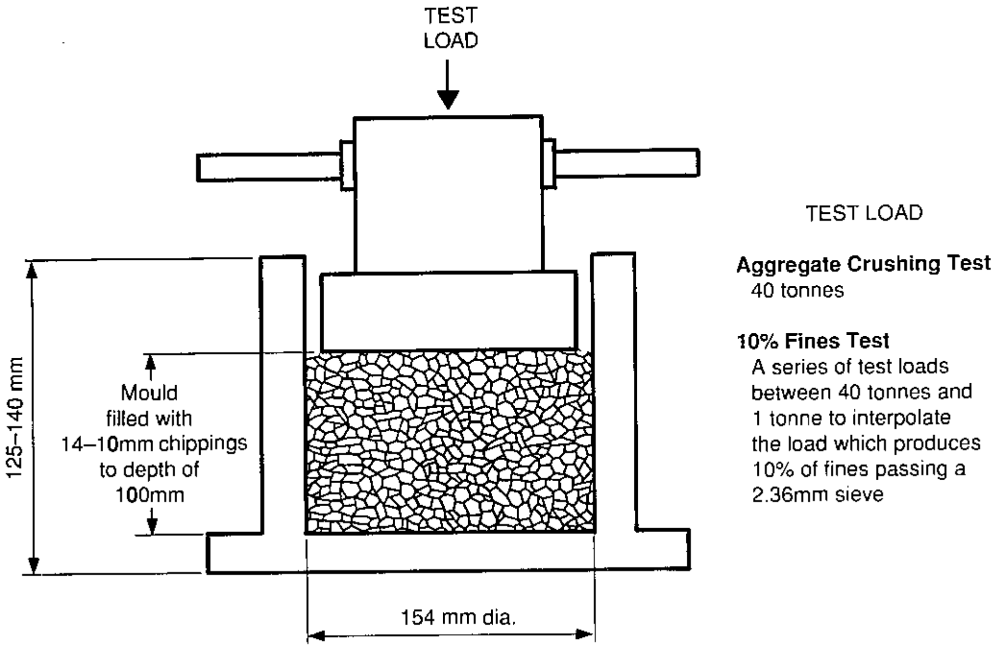


Fig.4.2 *Apparatus for the Aggregate Crushing Test and the Ten Per Cent Fines Test.*

4.4.3 *Aggregate Impact Test* BS 812 (1990), Part 112.

This test provides a simple and portable means of evaluating the strength of rock chippings. It employs a steel test mould with a falling hammer, as illustrated in Fig. 4.3. The Aggregate Impact Value (AIV) is the percentage of fines produced after 15 blows from the hammer. This test produces results that are normally about 105 per cent of the Aggregate Crushing Value and it can be used for the same purposes. Both tests give results which are sufficiently repeatable and reproducible for contract specifications.

4.4.4 *Los Angeles Abrasion Test* ASTM C131 and ASTM C535.

This test involves the use of a steel drum, revolving on a horizontal axis, into which the test sample of chippings is loaded together with steel balls of 46.8 millimetres diameter. The Los Angeles Abrasion Value (LAV) is the percentage of fines passing the 1.7 millimetre sieve after a specified number of revolutions of the drum at specified speed. The drum is fitted with internal baffles causing the aggregate and the steel balls to be lifted and then fall as the drum revolves. The test therefore gives an indication of the impact strength in combination with the abrasion resistance of the aggregate. For most aggregates the test results are similar to those obtained with the Aggregate Crushing Test. The repeatability and reproducibility of this test are satisfactory and appropriate for use in contract specifications.

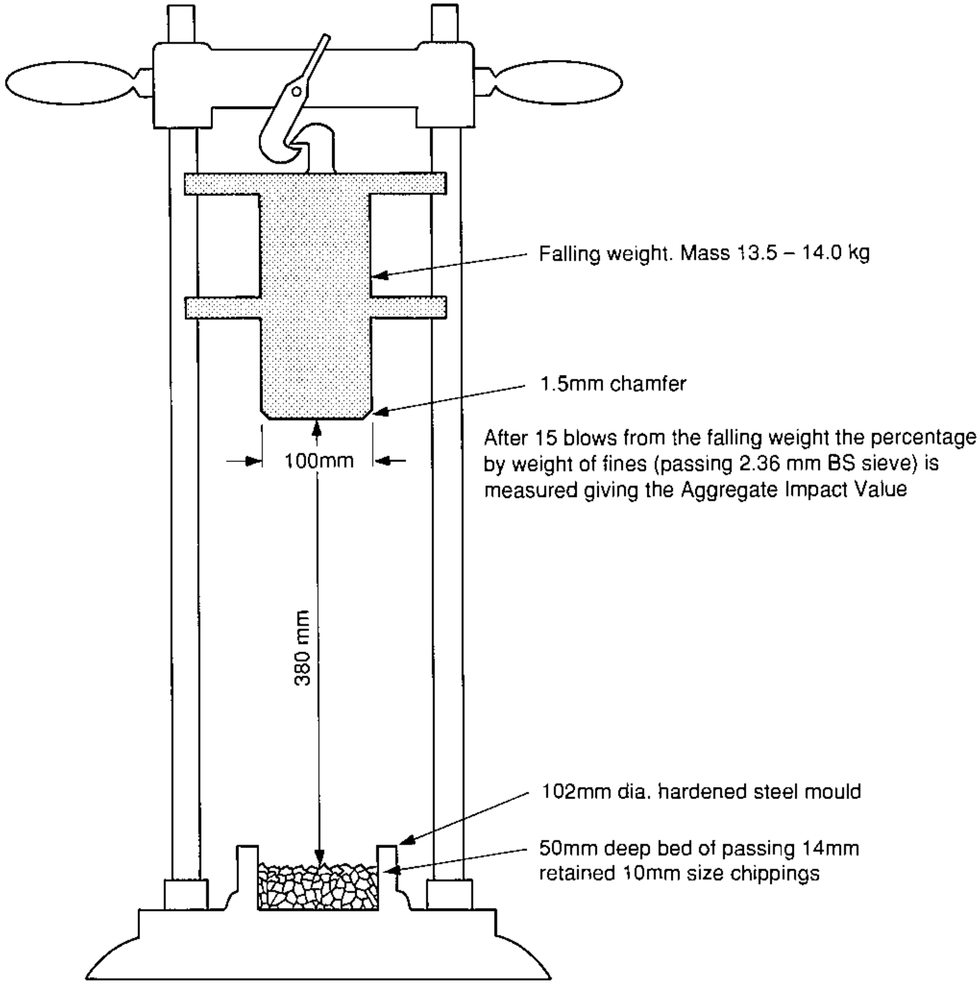


Fig.4.3 Apparatus for the Aggregate Impact Test.

The three tests, Aggregate Crushing, Aggregate Impact and Los Angeles Abrasion, give results that rank normal roadstones in a similar order of quality, and it is a matter of taste and convenience as to which should be used in given circumstances. None of the three is of use in detecting fine differences in tbe strength of the softer aggregates which may be the only ones readily available in some tropical countries. With soft aggregates, the Ten Per Cent Fines Test is likely to be more suitable, or the Aggregate Impact Test with a reduced number of blows. As part of the evaluation of such aggregates for use as roadbases, it may be more appropriate to examine the effects of standard compaction tests on their particle size distribution.

Criteria for the use of these tests are very much a matter for local experience. Indeed, road making processes often need to be adapted to make good use of the range of aggregates locally available and the chief value of the tests is in selecting the most suitable sources for quarrying. Other criteria will also be needed here e.g. resistance to polishing and affinity with bitumen (for aggregates to be used in bituminous surfacings), consistency in production, haulage costs etc. Within these limitations it will be the strongest aggregates that are preferred. This is particularly so for the aggregates used in bituminous surface dressing in which strong tough chippings are most desirable. For this use, chippings with an ACV less than 30 are desirable and the stronger they are the more durable will be the dressings. With premixed bituminous materials and with crushed stone bases, high mechanical strength, though useful, is not always of paramount importance.

4.4.5 *Aggregate Soundness* BS 812 (1989), Part 121; ASTM C88.

This test procedure is useful in both survey and design for the evaluation of aggregates suspected of chemical decomposition. Repeatability and reproducibility are poor. The test is not suitable for providing a single criterion for the susceptibility of aggregates to rapid weathering but it may find a place as part of the evaluation procedure of rocks suspected of containing minerals that are weakened by chemical alteration (see Section 4.2). Magnesium sulphate is preferred to sodium sulphate because of the greater penetrating power of the saturated solution.

4.4.6 *Specific Gravity and Water Absorption* BS 812 (1975), Part 2; ASTM C127.

The British and American methods are similar. The tests are likely to be used both in surveys of aggregate resources and in design, particularly in the interpretation of compaction tests and in the design of bituminous mixtures. They may also be used as part of quality control during construction, particularly when the survey has indicated that aggregate from the chosen source is subject to variations in density. The test procedure is simple and the tests are repeatable and reproducible. In some rocks there may be quite large variations in specific gravity. With lateritic gravels, for instance, there are likely to be quite large differences between the specific gravities of the coarse and fine fractions. In other concretionary materials and in metamorphic rocks, specific gravities may vary from point to point. In evaluating a deposit, it is desirable to test enough samples to determine both a representative average value and an indication of the likely range in values. Such testing is required during survey and design, but only in exceptional circumstances is it likely to feature in testing for compliance with specifications.

The specific gravity of natural rocks can vary over quite a wide range, between 2.3 and 3.4, and this variation is important in road engineering for two reasons. Its value must be known to determine the state of compaction achieved in crushed stone bases and in concrete and bituminous mixtures. Further, with both concrete and bituminous mixtures, wide differences from the average specific gravity (normally assumed to be 2.65) requires adjustments to the proportions of the ingredients in the mixtures. Although, in practice, they are proportioned by weight, it is the volume occupied by the different ingredients that is important in designing the mixtures.

Water absorption can also be important. Most rocks absorb less than one per cent by weight of water and, up to this level, water absorption is of no great consequence. However, some rocks can absorb up to 4 per cent of water. This suggests that the rock may be of low mechanical strength and that it may be susceptible to frost damage when used in roadbases in cold climates.

Furthermore, stone with a high water absorption will be difficult to dry and heat during processing to make bituminous mixtures. Inadequate drying will cause difficulty in securing good adhesion between bitumen and stone, and in hot process mixtures, where the stone must be heated to about 180°C, it hinders the heating of the stone. In a plant producing 1000 tonnes of asphalt per day, this could involve the evaporation of up to 4000 litres of water, a large waste of energy.

In the tests, a 4 kilogram sample of the crushed rock of specific nominal size chippings is soaked in distilled water for 24 hours, weighed in water, surface dried and weighed in air. It is then oven dried at 105°C for 24 hours and weighed again in air. The specific gravity is obtained by dividing the weight of the oven-dried sample in air by the apparent loss in weight of the saturated sample in water. The water absorption (percentage of dried weight) is obtained by expressing the difference in weights of the saturated and oven- dried samples in air as a percentage of the latter. Expressed in symbols, the equations are,

$$\text{Specific gravity} = \frac{W_D}{W_D - W_W} \quad , \text{Water absorption} = \frac{W_S - W_D}{W_D} \times 100\%$$

where W_W = the weight of the saturated sample in water
W_S = the weight of the surface-dried sample in air
W_D = the weight of the oven-dried sample in air

4.4.7 Affinity for bitumen

Rocks vary in their ability to form a permanent bond with bitumen. A high proportion of quartz can be indicative of poor affinity as can the presence of smooth faces such as in coarsely crystalline rocks and in broken flint and chert. Calcareous rocks bond well with bitumen. A reduced affinity for bitumen is never a disabling feature of a natural rock. It may be a weakness in some circumstances but there are simple techniques to secure good adhesion, reviewed in Chapter 5, and there is no reason for rejecting the use of any rock because of possible poor affinity with bitumen.

4.4.8 Resistance to skidding

In the United Kingdom's cool and damp climate, road surfaces are often wet. This has prompted a concern about accidents which involve skidding on wet roads. There is plenty of evidence to show that one of the reasons for the relatively low level of road accidents in the United Kingdom lies in the diligence with which road surfacings with a high resistance to skidding have been developed and used on more heavily trafficked roads.

In almost all developing countries, road accident rates are appallingly high and still rising. This has prompted much concern which has, so far, found expression mostly in words rather than effective action. One reason for this is that many road safety measures seem to be in conflict with the desire for cheap transport of people and goods. In those countries with wet climates, the provision of road surfacings which do not become slippery when they are wet can be an effective and inexpensive way of reducing the toll of road accidents.

In Barbados, road surfacings are made with limestone rock and the exposed aggregate can become highly polished under traffic so that the surfaces are quite slippery when wet. Whenever a rain storm occurs, traffic immediately slows down. In this island community, a pattern of road use has emerged which reduces the danger from wet, slippery roads. But this pattern is exceptional, perhaps even unique. Elsewhere, drivers seem to be quite unaware that, on a wet road, their capacity to control their vehicles in accelerating, braking and turning may be greatly impaired.

There are two important features in providing road surfaces which retain their non-skid properties when they are wet. On high speed roads the surfacing must have an open rough texture, perhaps even containing interconnecting voids so that the water film can be rapidly expelled from the contact area between the tyre and the road. On all roads where there is a risk of accidents involving skidding, the surfaces of the exposed aggregate must retain a sharp sand-papery texture so that a high level of friction can be developed between tyre and road. In dry weather, the presence of dust, oil droppings and even tyre-tread rubber promotes the polishing of the exposed aggregate under traffic, hence the phenomenon that roads are at their most slippery when rain falls on them after a prolonged dry spell. As the wetness continues, there can be some recovery in non-skid properties, but the important feature is that, with any particular aggregate, the degree of polishing is in direct proportion to the traffic intensity on the road. It is the busiest roads that become the most slippery when they are wet. It is therefore doubly important to provide a surfacing with a high resistance to polishing on the most heavily trafficked roads and especially on the approaches to busy junctions.

On concrete roads the rough macro-texture and micro-textures are produced by providing a ribbed surface at the time of construction. On old, smooth concrete this is done by cutting transverse ribs in the surface as illustrated in Fig. 10.9. On bituminous-surfaced roads the important feature is to use an aggregate with a high resistance to polishing in the surface. There is much more to this fascinating subject. Sabey (1966) has described the effects of using surfacings of high resistance to skidding in reducing the toll of road accidents, Salt (1977) has reviewed the selection of anti-skid criteria and Roe *et al.* (1991) have looked at the relationship between surface texture and road accidents on high speed roads. Here, our immediate concern is with evaluating the resistance of roadstones to the polishing action of traffic.

4.4.9 *Accelerated Polishing Test* BS 812 (1989), Part 114; ASTM D3319.

The American and British test procedures are identical. The test samples are prepared by mounting about 50 representative chippings in curved moulds using a sand-cement mortar (Fig 4.4). These are clamped round a 400 millimetre diameter wheel and are then subject to the polishing action of a revolving 200 millimetre diameter pneumatic tyre fed with abrasive powder (Fig. 4.5). The test runs for 6 hours. During the first 3 hours the abrasive powder is silica sand but for the second 3 hours a fine, air-floated emery powder is used. The state of polish of the sample is measured with a portable pendulum skid resistance tester (Fig. 4.6) and the results are reported as the Polished Stone Value (PSV).

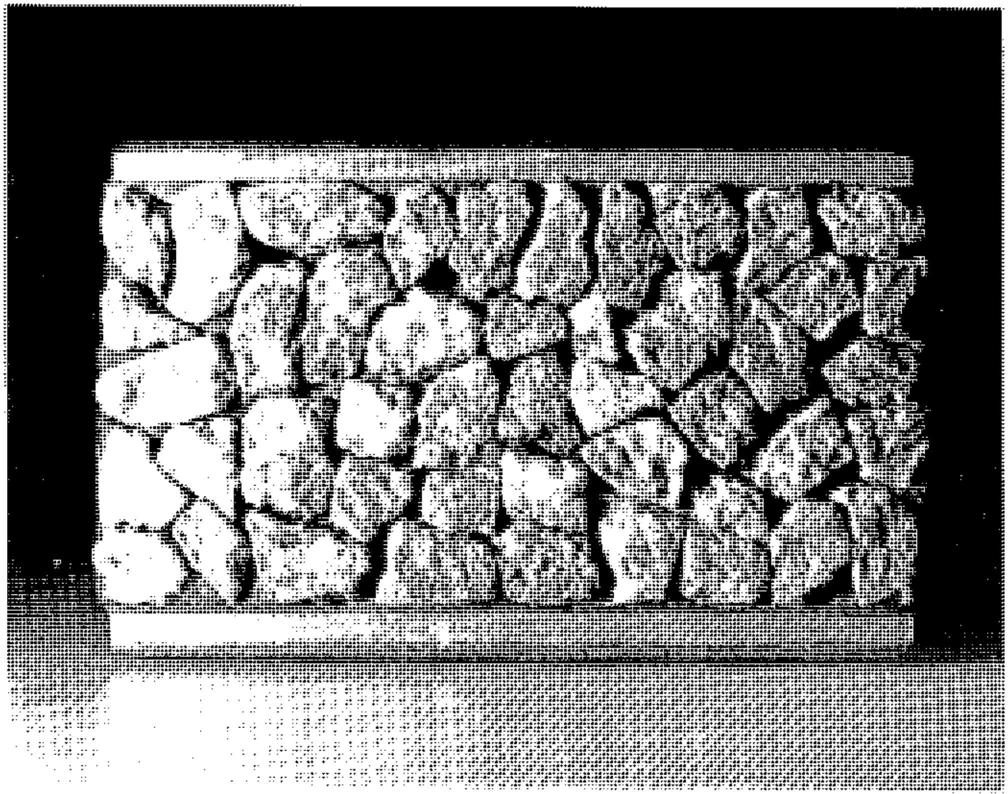


Fig.4.4 A sample mounted for the Accelerated Polishing Test.

The test result can be regarded as a coefficient of friction. It ranges from 0.3 for stone likely to acquire a high degree of polish to 0.8 for a stone likely to retain its gritty surface texture under the heaviest traffic.

It is a relatively complicated and tedious test and is likely to be done only in the central laboratories of highway authorities and specialist testing consultants. It is undertaken in examining the potential of new sources of roadstone where a high resistance to skidding on wet roads is judged to be important. Minimum Polished Stone Values may be specified for use in bituminous surfacings on more heavily trafficked roads. Under these circumstances, quarry operators find it an advantage to obtain certification of the PSV of their stone from testing laboratories which possess the testing equipment and are familiar with its use.

Some generalisations can be made about the propensity of different types of stone to polish under traffic. Limestones are generally highly prone to polish, those that contain increasing proportions of silica being less prone. Gritstones normally have a high resistance to polishing and, amongst the igneous rocks, there is a trend for basic rocks to polish more readily than acid rocks. The most resistant of all is calcined bauxite, a specially prepared aggregate used in proprietory surfacings where an exceptionally high resistance to skidding is required.

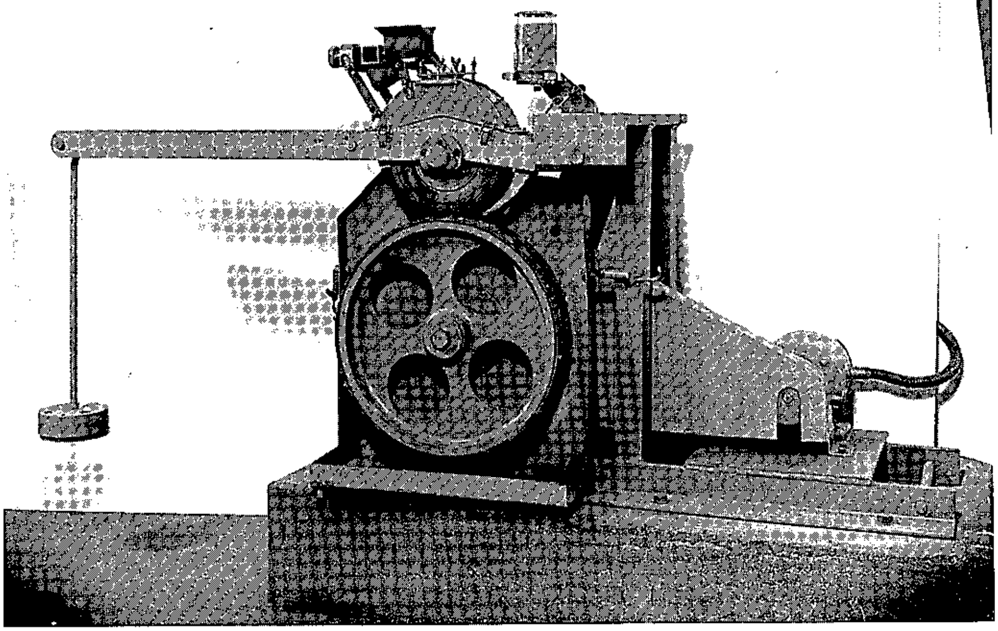


Fig.4.5 Apparatus for the Accelerated Polishing Test.

The danger of skidding is not likely to be important in dry climates except, possibly, when there is a thin layer of dry wind-blown sand on the surface. Even in climates where the rain comes as occasional storms, road surfaces tend to dry quite quickly in hot weather and the exposure to wet road skidding accidents may not be extensive. But there are some regions where the seasonal cycle includes periods of persistent high humidity during which there can be a high risk of skidding accidents on busy roads. West Malaysia is one example where the substitution of an igneous rock in place of local limestones in the bituminous surfacings on more heavily trafficked roads has helped considerably in reducing the toll of road accidents. The range of PSVs obtained with these two aggregates has been reported by Beaven and Tubey (1978).

Police records of accidents may be helpful if they include reliable information on whether the road surface was wet or dry at the time of accidents. Clusters of accidents occurring at junctions can be an indication that improvements are needed to the road layout but, as Hatherley (1978) reports, quite phenomenal reductions in accident rates at junctions were obtained in London by the use of special anti-skid surfacings on the approaches to the junctions.

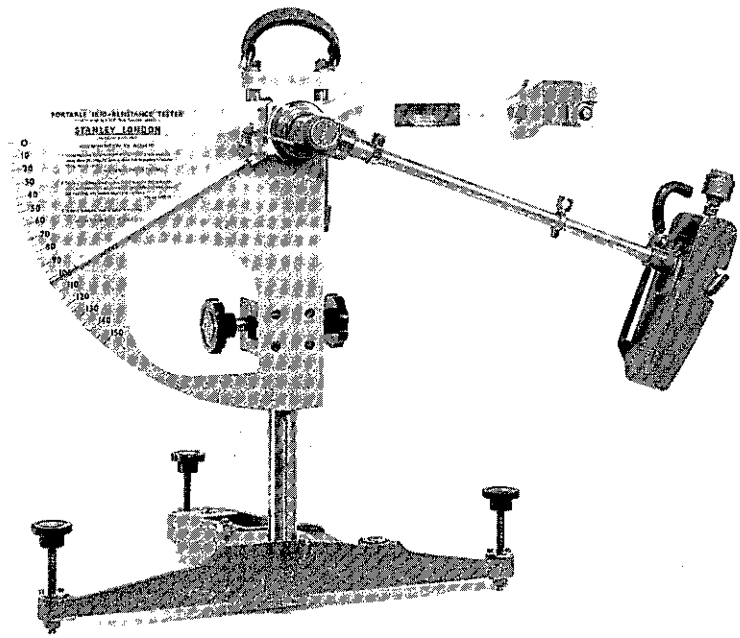
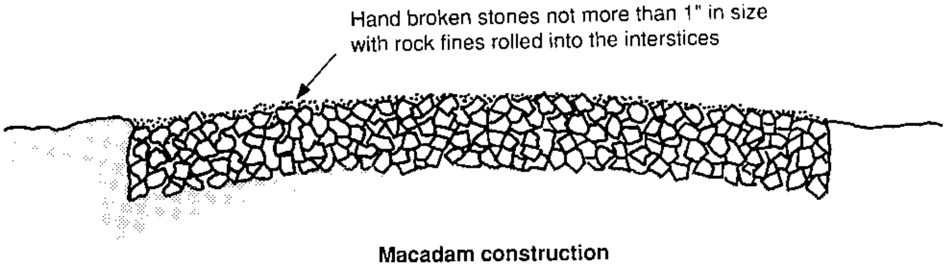


Fig.4.6 Pendulum skid resistance tester.

4.5 Testing of processed aggregates - size and shape

The previous test procedures have been concerned with the intrinsic properties of rocks. Now we come to properties which derive from the way in which the rocks are processed to produce useful road making materials i.e. the crushing and screening processes. It is appropriate to start with one of John Loudon Macadam's precepts that no stone used in making a road should be larger than one inch in diameter. This was said in the days when rocks were laboriously broken by hand, a form of poor relief still much in evidence in the Indian sub-continent. Macadam's precept remains substantially true today in an era when the quarrying of stone and other road making processes are heavily mechanised. Telford's alternative method of making a road base, with small boulders coarsely dressed to the shape of elongated pyramids and keyed in place with small stones hammered in from above, was still in use within living memory but it was painstakingly slow and required the skills of a mason to build the tight, flat arch which was essential to the strength and stability of the pavement. These two traditional methods of building pavements are illustrated in Fig 4.7.

Modern road building methods have added a further requirement that the crushed stone must be available in a variety of specific sizes that can be used either individually or recombined to make mixtures with specific ranges of particle size suitable for making asphalt, concrete and dry stone



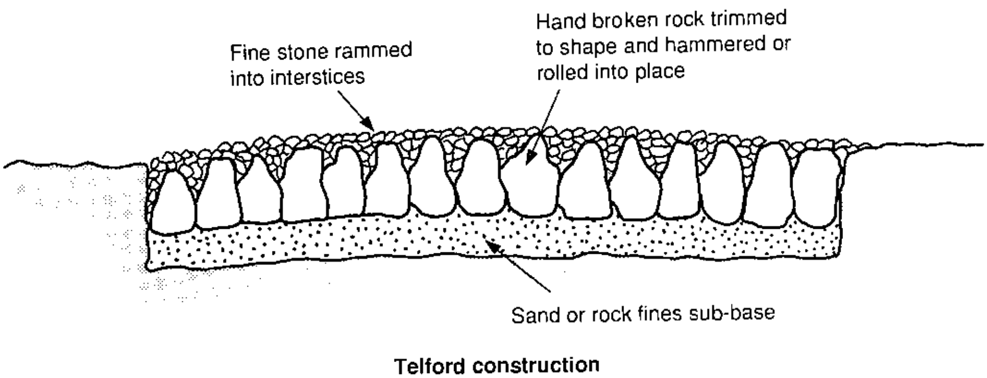


Fig.4.7 *Traditional methods of road pavement construction.*

roadbases. Further, the demand is for chippings of roughly cubical shape, avoiding flaky or elongated particles which can be a source of weakness in the final product.

4.5.1 *Particle Size Analysis* BS 812 (1985), Part 103; ASTM C136; AASHTO T27.

There are minor differences between the sieve sizes in the American and British specifications.

British:		
Square hole:	millimetres,	75, 63, 50, 37.5, 28, 20, 14, 10, 6.3, 5.0
Wire cloth:	millimetres,	3.35, 2.36, 1.70, 1.18,
	micrometres,	850, 600, 425, 300 212, 150, 75
American:		
Coarse:	millimetres,	150, 120, 112, 100, 90, 75, 60, 50, 37.5, 25, 19, 12.5, 9.5
Fine:	millimetres,	4.75, 2.36, 1.18
	micrometres,	600, 300, 150

A sedimentation test is also included for the determination of clay and silt content.

The results are reported as either the total percentage by weight passing each sieve or, in the case of single-sized aggregate, the percentage by weight of the specified size and the percentages of oversize and undersize material.

If sieving is done by hand, the sieves must each be shaken for at least two minutes, taking care not to overload the sieves. Nowadays, mechanical sieving machines are generally available. These are often heavily used and it is important to inspect the sieves regularly, replacing any that show signs of wear.

4.5.2 *Particle Shape* BS 812 (1990), Part 105; ASTM D3398.

The Standards designate similar methods for determining the proportion of flaky and elongated particles in a sample. As with particle size analysis, these tests are undertaken for quality control purposes during construction. Particle size and shape tests will be undertaken as part of the procedure for approving a particular supply of aggregate and they should be repeated at regular intervals during construction to make sure that specified values are consistently maintained.

Flaky particles are defined as those having their smallest dimension less than 0.6 of their mean size, and elongated particles as those having their largest dimension more than 1.8 times their mean size. For this purpose, mean size is defined as the mean of the two aperture sizes between which the particle is retained. Examination for flakiness and elongation is normally done on one nominal size of chipping. With graded aggregates, this is on the fraction passing the largest sieve size and retained on the second. The percentage by weight of flaky and elongated chippings is recorded. The percentage permitted remains a matter for debate and guidance can be obtained from the current edition of the appropriate standard.

These two tests may he used at the design stage to check on the quality of available crushed stone. Their main use is in checking that the grading and particle shape of rock chippings conform with specifications for particular works. Limits of accuracy are not quoted in the Standards but, provided that sampling is properly done, the tests are sufficiently reproducible for use in contract specifications. Such specifications should always refer to samples obtained either in the quarry or on delivery to the site i.e. before they have been subject to the stressing of compaction and subsequent traffic. Some degradation is likely under such stressing, even with the hardest of stones.

4.6 Quarries

Some 290 million tonnes of crushed rock, gravel and sand were processed in Great Britain in 1988 i.e. about 4 tonnes per head of population. Of this amount, some 90 million tonnes was used in road construction and maintenance, 32 million tonnes of which were supplied as bituminous-coated material. Almost all was produced in permanent rather than temporary quarries and pits.

At the other extreme, we have the situation in some developing countries where, in more remote areas, road building and other civil engineering works require temporary quarries to be established which are often abandoned once the construction works are completed. After they are gone, the area is bereft of good crushed aggregate for use in road maintenance and other works. In West Malaysia, moves were made in the late 1950's to change this situation. The Public Works Department constructed a chain of about 20 permanent quarries so distributed that crushed stone was available within 80 kilometres over almost the whole area of the country. This effectively transformed the ability of the Public Works Department, enabling it to embark on an effective programme of highway rehabilitation and new construction. It also provided sources of good quality aggregates for other civil engineering purposes.

Where rock deposits are remote, the crushed rock is often carried by water. An example is in Bangkok, where many of the barges coming down river are fully loaded with crushed stone. A very large quarry has been recently commissioned on the west coast of Scotland from which crushed stone is conveyed to ports in Northern Europe and across the Atlantic to the eastern seaboard of North America.

A comprehensive description of modern quarrying methods is contained in 'Aggregates' a publication of the British Geological Society (see Collis and Fox (1985)). The purpose here is to outline the quarry practices needed to produce sound crushed stone particles of good shape and in the sizes required for the range of road making processes.

Small scale production is common in many developing countries. In Sri Lanka, for instance, its presence is announced by the notice METAL FACTORY at the side of the road. Through the trees can be seen a small quarry face, and a track by the notice leads to a small compound with stone-crushing equipment in one corner. This equipment consists of a small jaw-crusher and screens made by the local blacksmith, all driven from a small engine through a rickety set of belts. The stone is extracted from the quarry face and broken down by hand to be fed into the crusher. It is collected from below the screens in wheel- barrows. The effort is heroic but the output is pitifully small and of indifferent quality. Here are people who, with encouragement, could become quarry masters on a larger and more effective scale.

To produce good quality road aggregates it is necessary to employ at least two stages of crushing in order to reduce the rock from the quarry face to suitable sizes. Modern quarry installations have three or more stages of crushing. This provides the needed flexibility to alter crusher settings and screens so that the quantities of stone of different sizes can be adjusted from time to time to meet changes in demand. Quarry installations vary considerably in the arrangement of their different components. A typical arrangement of a three-stage crushing installation is shown in Fig. 4.8.

The rock boulders delivered to the primary crusher pass first over a scalping screen to remove dirt and weak rock which has already fragmented during removal from the face. The primary crusher will normally be a large jaw-crusher producing an output of material up to 250 millimetres in size.

Secondary and tertiary crushing is done with jaw, cone, disc, gyratory, hammer or roller crushers, the type being selected according to the nature of the rock. The essential features in producing clean, strong rock chippings of good shape include a low reduction ratio, usually less than four at each stage in crushing, removal to waste of fine dirty material, both before and after primary

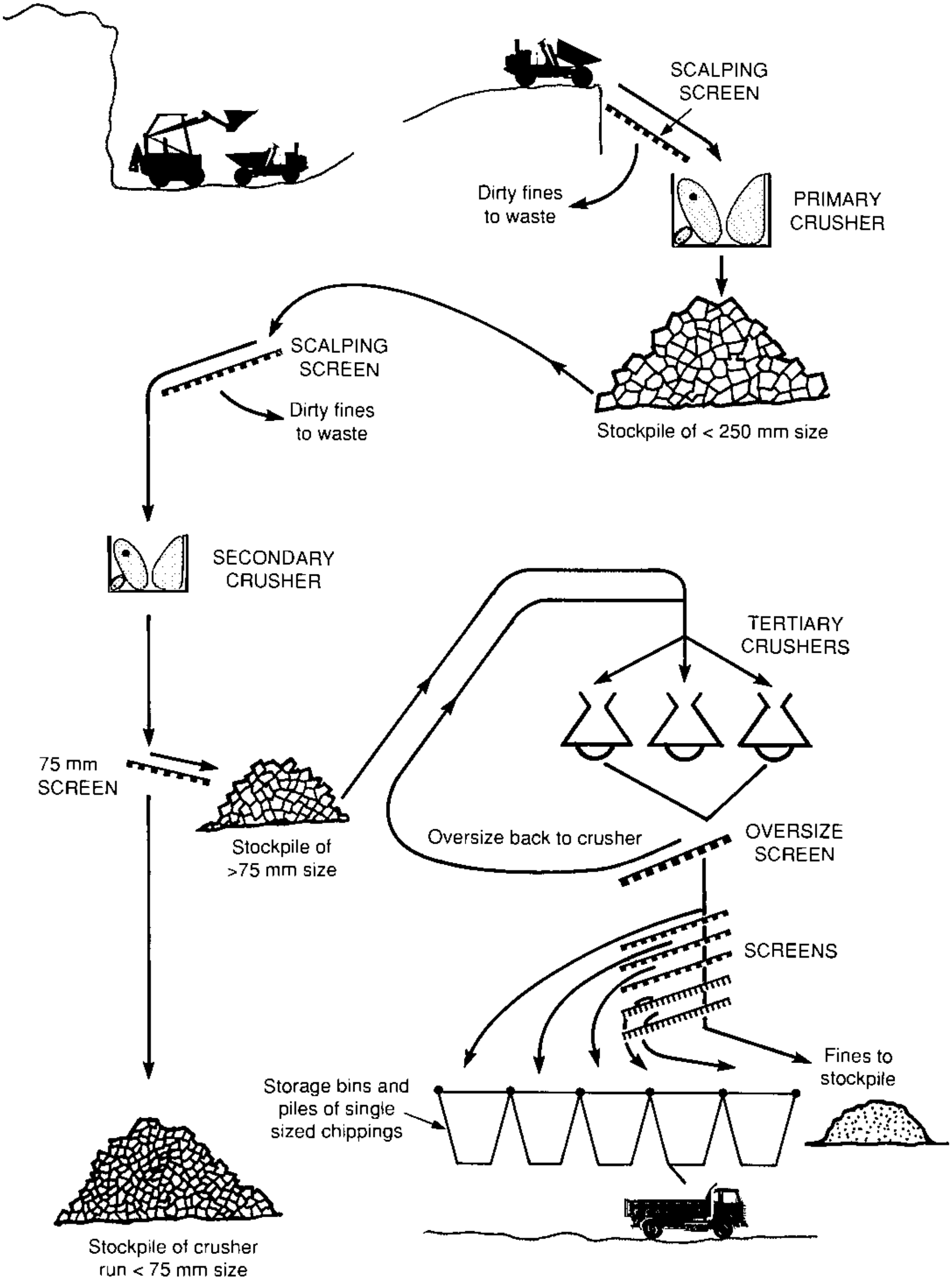


Fig.4.8 Layout of medium size crushing and screening plant.

crushing, and choke feeding of secondary and subsequent crushers. To these should be added vigilance in maintaining the equipment in good working order. Crusher faces and screens are subject to heavy wear and they need to be replaced as soon as signs of wear are evident. Adjustments must be made, often quite frequently, to crusher settings and screens to keep the output of single-sized chippings in step with the demand for particular sizes. One of the main aims in quarry management is to maintain a balanced output consistent with demand. This requires considerable skill and can be very rewarding to the effective quarry operator.

Unduly high reduction factors are the commonest cause of flaky chippings. Some rocks, particularly fine-grained igneous and metamorphic rocks, have a reputation for producing flaky chippings. There is some justification for this but any such trend can be controlled within reasonable limits by maintaining a low reduction factor at each stage of crushing.

For an adequately equipped quarry to be commercially viable, there needs to be a fairly stable demand, probably for at least 25,000 tonnes of its annual output. Demands far in excess of this occur in the more populous areas of nearly all countries. It seems likely that one of the most helpful subjects for financial aid and technical assistance is the provision in such areas of the quarries that are necessary to process a local resource which is used in every form of infrastructure development.

Many quarries will have asphalt plants installed to provide bituminous mixtures for building, maintaining and strengthening roads. Although the use of such plants may sometimes be intermittent, there are great advantages in maintaining a corps of operators skilled in the mixing and laying of good quality asphalts.

Crushed aggregates are also needed in areas remote from centres of population, not only for building and maintaining roads but also in building water supply and irrigation works and, frequently, also in harbour works in coastal towns and villages. The demand may often be insufficient to justify a well equipped quarry and therefore the local people may have to make do with small quarries containing quite rudimentary crushing and screening equipment, making special arrangements for screening out the particular sizes of stone needed for specific purposes e.g. for making concrete and for patching and repairing potholes on roads. Robinson (1979) has described manual methods of screening which can be useful in labour intensive works.

There are other ways to cope with this situation. For example, the quarry equipment and experienced personnel brought in for major works could be used to build up stocks for future use in the area. After building a new road, it is fairly certain that much of its length will need to have a bituminous surface dressing within the first 10 years. Even with the opportunity cost of capital at over 10 per cent, it will be worth building up a stock of the single-sized chippings needed for such work. It may be possible to leave the quarry equipment in place after its capital costs have been written off against the work for which it was bought, operating it by either a local contractor or district public works department. Alternatively, teams could be sent to the plant every few years to build up stocks of crushed aggregate for local use.

Breaking stone by hand is still common practice in many parts of the Indian sub-continent. It provides work for people who have little opportunity to find other gainful employment. Both there and in other parts of the world where local self-help schemes are in operation, it may be the only way to obtain broken rock for use in making roads and buildings, but it is a slow and tedious process. The sizes of broken rock which are produced, namely cobbles of 30-60 millimetres diameter plus fine chippings, are appropriate for the traditional macadam construction (Fig. 4.8) in which the fines are used to fill the voids in the closely packed cobbles. They are also still used to make grouted macadam, an early form of bituminous pavement. In this process, hot bitumen or bitumen emulsion is poured into the voids of the compacted cobbles to a depth of 80-100 millimetres before the final application of fines. When well done, this makes a tough and flexible road pavement but, compared with modern methods, it uses rather a lot of bitumen. Breaking stone by hand cannot produce chippings in the quantity and of the quality required for making bituminous surface treatments, asphalt surfacings and durable concrete in any quantity.

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5 Bituminous binders

5.1 Introduction

Bitumen is a black sticky substance derived from processing geological deposits of crude oil. In many parts of the world it can be found as a component of natural asphalts. These are rocks or soils originally impregnated with oil from which the more volatile constituents have evaporated. Some of these natural asphalts can be used in road making but by far the largest supplies of bituminous binder in commercial use are derived from the refining of crude oil in the production of petrol, diesel oil, lubricating oil, etc. It is somewhat providential that the material which provides the fuel for motor vehicles also provides the binder used in making the roads on which the vehicles run. There is also road tar derived from the distillation of coal in the production of coke and coal gas. Road tar has somewhat different properties from bitumen and its use on roads has been declining, partly because of the valuable chemical compounds that can be extracted from it.

It is as well to be aware of a confusion in terminology between the New World and the Old. In Europe, the black sticky substance derived from crude oil is bitumen (bitume in French and German, bitum in Spanish) and mixtures of bitumen and stone, whether natural or man-made, are asphalts. In North America, bitumen is called asphalt or asphaltic cement and there is no general term for bitumen-stone mixtures unless the ubiquitous type of bituminous surfacing, asphaltic concrete, can be regarded as such.

Not all crude oils are bituminous. Those from Indonesia, Nigeria and the North Sea, for instance, are light crudes containing very little bitumen. Heavy crude oils, such as those from Central America and the Middle East, usually have quite a large bitumen content. Some crude oils are paraffinic and the paraffin wax contained in the bitumen reduces its value for road making.

There are small variations in the nature of the bitumens from different oil fields, even between wells in the same field, but the major bitumen suppliers are careful over the quality of their product, modifying the production process so that these variations are reduced in the interests of supplying a uniform product. This is known as refined bitumen i.e. the product of a refinery, or sometimes as residual bitumen i.e. the product remaining after the lighter fractions of the crude oil have been removed by distillation. Brief descriptions of the properties of bitumen and of the commercial forms in which it is supplied are given in this chapter. There is a vast literature on the subject and readers who wish to know more are referred to a recent handbook published by Shell Bitumen UK Ltd (see Whiteoak, 1990) and the Asphalt Handbook published by the Asphalt Institute (1989).

5.2 Rheological properties of bitumen

Bitumens become fluid when heated and in this condition they can be readily used in various road making processes. The viscosity-temperature characteristics of typical commercial bitumens used in road making are shown in Fig. 5.1. On the log-log scale shown, the relationship is linear. A kink in the lines in the region of 80°C suggests a bitumen made from a paraffinic crude i.e. containing paraffin wax. Waxes have a defined melting point at which they change abruptly between the solid and liquid phases. There is no such abrupt change with bitumens that are not paraffinic. As they cool, they stiffen gradually from a fluid condition to become a tough solid material at ambient temperatures, with visco-elastic properties. The elastic component is an important feature. It means that they can absorb sudden strains without cracking.

The visco-elastic properties of bitumens are well illustrated by the Burgher's model shown in Fig. 5.2. Under constant loading, there is an initial deformation in the springs and, thereafter, a continuing deformation in the dashpots. When the load is removed, the energy stored in the springs is discharged by reverse deformation in the dashpots, as shown in Fig. 5.3. Under transient loading, the springs deflect, helping to reduce the stresses in the material below a critical level. This visco-elastic behaviour is shared to some extent by asphalt road surfacings and bases; it explains the use of the term 'flexible pavement' to describe roads with bituminous surfacings. They can move under slow strains without cracking and they have a built-in ability to absorb the energy of impact loads such as those imposed by traffic. It also explains the superior resistance to cracking of surfacings in which the bitumen is present in relatively thick films, such as in the mastic asphalt used on bridge decks, which contains up to 18 per cent of bitumen, and in surface dressings in which a relatively thick film of bitumen is used to hold the stone chippings to the road surface, as illustrated in Fig. 5.4.

5.3 Refined bitumens, cutbacks and bitumen emulsions

Refined bitumen is used in making asphalts. Both the bitumen and the aggregate are heated to a relatively high temperature, high enough to make sure that the aggregate is completely dry and the bitumen sufficiently fluid before the two ingredients are mixed together. For some processes it is either not convenient or not possible to heat the aggregate, for example, in surface dressing, in soil stabilisation, and in priming a surface before covering it with new asphalt. For such processes, cutback bitumens and bitumen emulsions are used.

With cutbacks, the bitumen is fluxed with a lighter oil fraction so that the binder can be applied either at ambient temperature or heated to relatively low temperatures, usually less then 150°C. Cutback bitumens start to harden by the volatilisation of the fluxing oil immediately after they are put to use. The rate of hardening is determined by the volatility of the fluxing oil. Where rapid hardening is needed, as in surface dressing, a light oil such as kerosene is used. Delayed hardening is needed in making patching materials and in soil stabilisation. Slow curing cutbacks are made with a proportion of less volatile oil such as diesel oil. Cutbacks for cold application may contain up to 30 per cent of fluxing oils. More viscous cutbacks, which are applied warm, contain less fluxing oil.

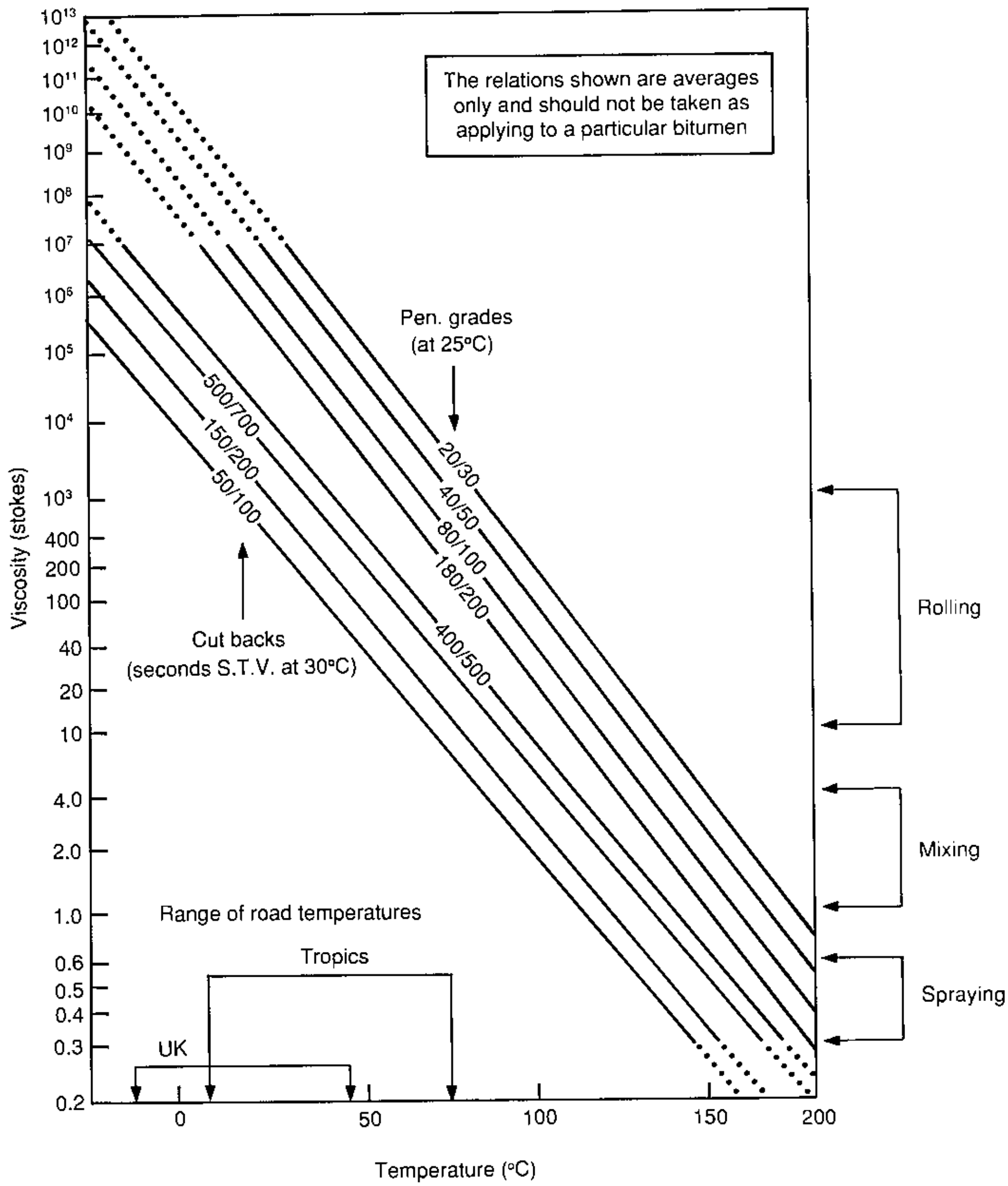


Fig.5.1 *Viscosity - temperature relationships for road bitumens.*

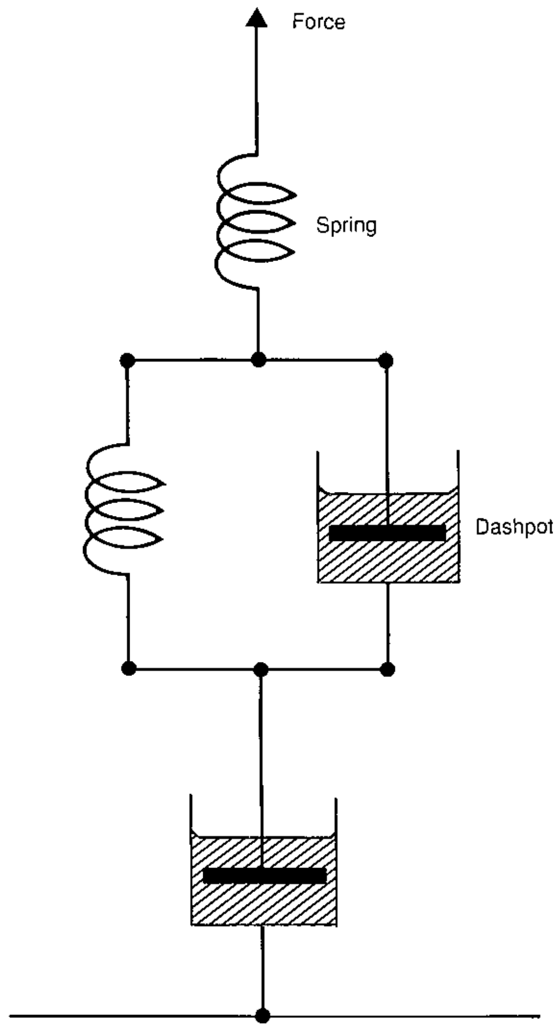


Fig.5.2 *The Burgher's model for deformation in a visco-elastic material.*

Bitumen emulsions are dispersions in water of minute drops of refined bitumen of average diameter about 2 microns. Their purpose is to provide a means of using refined bitumen at ambient temperature. Emulsions are quite common in nature. Milk, for instance, is an emulsion of carbohydrates in water. Under gentle heating, the water evaporates leaving clotted cream. With bitumen emulsions, the water evaporates shortly after the emulsion is exposed to the atmosphere. Bitumen emulsions are muddy brown in colour, and the 'breaking' of the emulsion into its constituent parts is signalled by the appearance of a continuous glossy black film of bitumen on the surfaces to which the emulsion is applied.

Emulsifying agents are used to produce such dispersions of bitumen in water and to control their persistence. Colloid mills are employed in the manufacture of emulsions. In such mills, a cylindrical rotor revolves at very high speed with an extremely small clearance between its

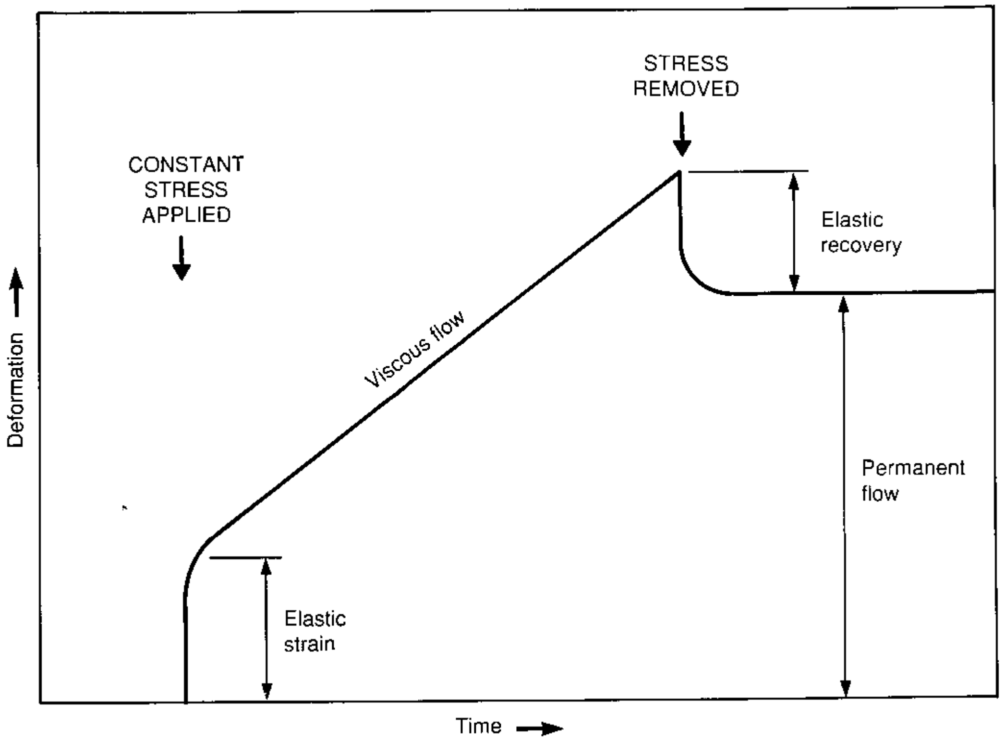


Fig.5.3 *Deformation as a function of time for visco-elastic material.*

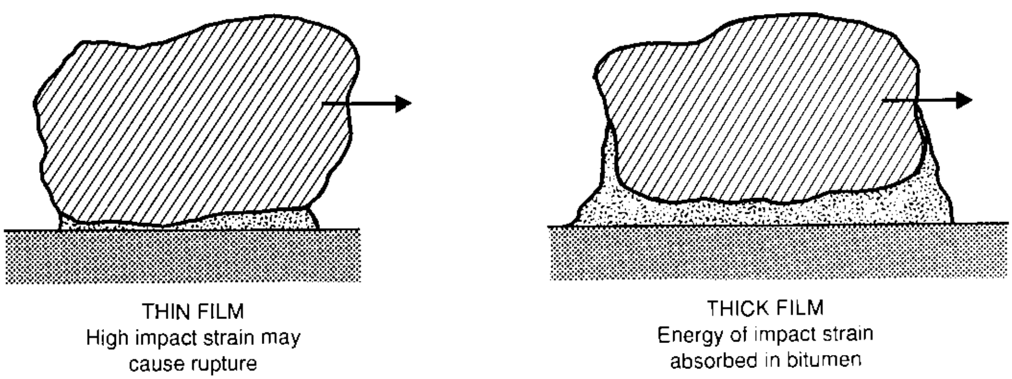


Fig.5.4 *The effect of bitumen film thickness in absorbing impact strain.*

surface and the cylindrical stator within which the rotor revolves. Hot water containing the emulsifying agent and hot bitumen are fed separately into the mill and the emulsions are produced by the very high shearing stresses between the stator and the rotor. The water content is varied, usually between 30 and 60 per cent, according to the properties and uses intended for the emulsion.

Casein and fine clay were once generally used as emulsifying agents. They are still used in bitumen emulsions for some industrial purposes, but in road making, they have been replaced by the anionic and cationic emulsifiers. Anionic emulsifiers were the first on the scene. These are soaps in which each particle of bitumen in the emulsion is surrounded by an absorbed film of stearate ions with an external negative electrical charge, a charge which inhibits the particles from coalescing. This negative charge has some effect on the use of the emulsion in road making. It assists the bitumen in bonding to aggregates which have a positive electrical charge on their surfaces e.g. limestones and other rocks containing calcium and magnesium carbonate. Cationic emulsions followed. The emulsifying agents in these are organic compounds in which the cation component of the molecule is soluble in bitumen leaving a positive charge on the surface of the bitumen droplets, again serving to discourage coagulation. In addition, it assists in bonding with more acidic rocks which carry a negative electric charge on their surface e.g. rocks containing a high proportion of quartz. These organic compounds, often long-chain amines, are also used to promote adhesion between refined or cutback bitumens and stone, whenever this is necessary.

Cutback bitumens are becoming less popular because of the value of the fluxing oils that they contain and which are lost into the atmosphere when the cutbacks are used. But they have the advantage that they can be manufactured in the field, for instance, when small quantities are required for road maintenance and the manufacture of patching materials. Care must be taken in proportioning the ingredients and, particularly, in avoiding the risk of fire. Methods for the manufacture of such cutbacks are described by Hitch and Stewart (1987).

At one time, bitumen emulsions were unpopular because of the costs of transporting a material of which up to 60 per cent is water. Their popularity has increased considerably with the advent of anionic and cationic emulsions and they have particular advantages in developing countries because they do not need heating and they can be applied using quite simple equipment. The manufacture of emulsions requires considerable expertise and skill. Adjustments may be needed to the acidity of the bitumen, and slight variations in the purity of the water can affect the behaviour of the emulsions. It is a task best left to experts.

Refined bitumens, cutback bitumens and bitumen emulsions are all the subjects for National Standards. Before continuing further with descriptions of the properties and uses of bituminous materials it is useful to review these Standards together with the tests which are used to define the characteristics of the three forms of bitumen which are supplied for use on roads.

5.3.1 Refined bitumens

BS 3690 (1989).	Bitumens for building and civil engineering. Part 1. Bitumens for roads and other paved areas.
ASTM D946.	Penetration-graded asphalt cement for use in pavement construction.

The British and American standard grades are described in Table 5.1. The consistency of the bitumen is defined by means of two tests, the Penetration Test and the Softening Point Test. In practice, the different grades are designated by their Penetration. The hardest grade, 15 pen., is used for making mastic asphalt for surfacing bridge decks. Those between 40 pen. and 200 pen. are used for making asphalts and bitumen macadams. In most European countries, bulk supplies are available over the whole range of penetration grades. In many Third World countries, only one grade is available for bulk delivery, usually 100 pen. Other grades have to be specially ordered for delivery in drums.

For routine testing of deliveries it is normal to check only the Penetration and Softening Point. The other designated tests are undertaken when new sources of supply are being evaluated or in other special circumstances. The appropriate Standards describing these tests are:

Penetration Test	BS 2000 (1983), Part 49; ASTM D5 and IP 49.76
Softening Point Test	BS 2000 (1983), Part 58; ASTM D36 and IP 58.65

The IP references are to the Handbook of the Institute of Petroleum, an organisation which has the prime responsibility for designating and revising tests on oil products in Great Britain. The British and American test procedures are similar but not identical. Softening Points determined by the ASTM method are 1.5°C higher than those produced by the IP method.

The Penetration Test is illustrated diagrammatically in Fig. 5.5. A standard needle is set just touching the surface of a sample of bitumen in a water bath at 25°C. The needle, loaded with 100 grams, is then released for exactly 5 seconds and the Penetration is recorded as the depth in tenths of a millimetre that the needle penetrates into the bitumen. With early penetrometers it was necessary to have two steady hands and it was an advantage to be cross-eyed in order to observe both the sample and the stop watch. Nowadays the test is automated so that it is more accurate and easier to perform. A modern penetrometer is illustrated in Fig. 5.6.

The Softening Point Test is illustrated diagrammatically in Fig. 5.7. In this test, two discs of bitumen, contained in a standard mould, are mounted in a water bath with steel balls placed over the centre of each disc. The temperature of the water is increased at a steady rate of 5°C per minute and the Softening Point (Ring and Ball) is defined as the temperature at which the steel balls fall through the bitumen to touch the plate below.

Starting afresh it might be better to define the consistency or viscosity of bitumens at temperatures which are of significance in their use; for instance, one test for indicating the temperature at which they become sufficiently fluid for mixing or spraying and another to define the consistency at road temperature. In Australia, moves are being made in this direction, as described in Australian Standard AS 2008 and by Dickinson (1984). But the Penetration Test and the Softening Point Test are well entrenched, with such lore behind them that they are not likely to be generally superseded. Part of this lore lies in the Penetration Index (PI), a relationship between the Softening Point and Penetration initiated by Pfeiffer and van Doormal (1936), which indicates the temperature susceptibility of the bitumen i.e. the slope of the viscosity - temperature relationships in Fig. 5.1. A nomograph indicating how the PI can be derived is shown in Fig. 5.8. Normal refined bitumens have a PI between -1 and +1. Bitumens with a PI of 3 and above are more rubber-like i.e. the elastic component of deformation under load is increased. Such bitumens are produced by blowing air through the bitumen at high temperatures. These are

TABLE 5.1
BRITISH AND AMERICAN GRADES OF REFINED BITUMEN

(a) **British (BS 3690. 1989 Part 1)**

Property	Test method	Technically identical with	Grade									
			15 pen	*25 pen*	*35 pen*	*40 pen HD**	*50 pen*	*70 pen*	*100 pen*	*200 pen*	*300 pen*	*450 pen*
Penetration at 25°c	BS2000 Part 49	ASTM D 5 IP 49/76	15± 5	25±5	35±5	40±10	50±10	70±10	100±20	200±30	300±45	450±65
Softening point °C												
(min)	BS 2000	ASTM D 36	63	57	52	58	47	44	41	33	30	25
(max)	Part 58	IP 58/65	76	69	64	68	58	54	51	42	39	34
Loss on heating for 5 hr at 163°C	BS 2000 Part 45	IP 45/58										
(a) Loss by mass % (max)			0.1	0.2	0.2	0.2	0.2	0.2	0.5	0.5	1.0	1.0
(b) Drop in penetration % (max)			20	20	20	20	20	20	20	20	25	25
Solubility in trichloroethylene % by mass (min)	BS 4690	IP 47/74	99.5	99.5	99.5	99.5	99.5	99.5	99.5	99.5	99.5	99.5
Permittivity at 25°C and 1592 Hz (min)					2.630	2.650	2.650	2.650				

*A special heavy duty bitumen used in making asphalt bases of enhanced stiffness.

(b) **Amercian ASTM. D946**

	Penetration Grade									
	40-50		*60-70*		*85-100*		*120-150*		*200-300*	
	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
Penetration at 77°F (25°C) 100g, 5 s	40	50	60	70	85	100	120	150	200	300
Flash point °F (Cleveland open cup)	450	*	450	*	450	*	425	*	350	*
Ductility at 77°F (25°C) 5 cm/min, cm	100	*	100	*	100	*	100	*	100	*
Solubility in trochloroethylene, %	99.0	*	99.0	*	99.0	*	99.0	*	99.0	*
Retained penetration after thin-film oven test, %	55+	*	52+	*	47+	*	42+	*	37+	*
Ductility at 77°F (25°C) 5 cm/min, cm after thin film oven test	*	*	50	*	75	*	100	*	100	*

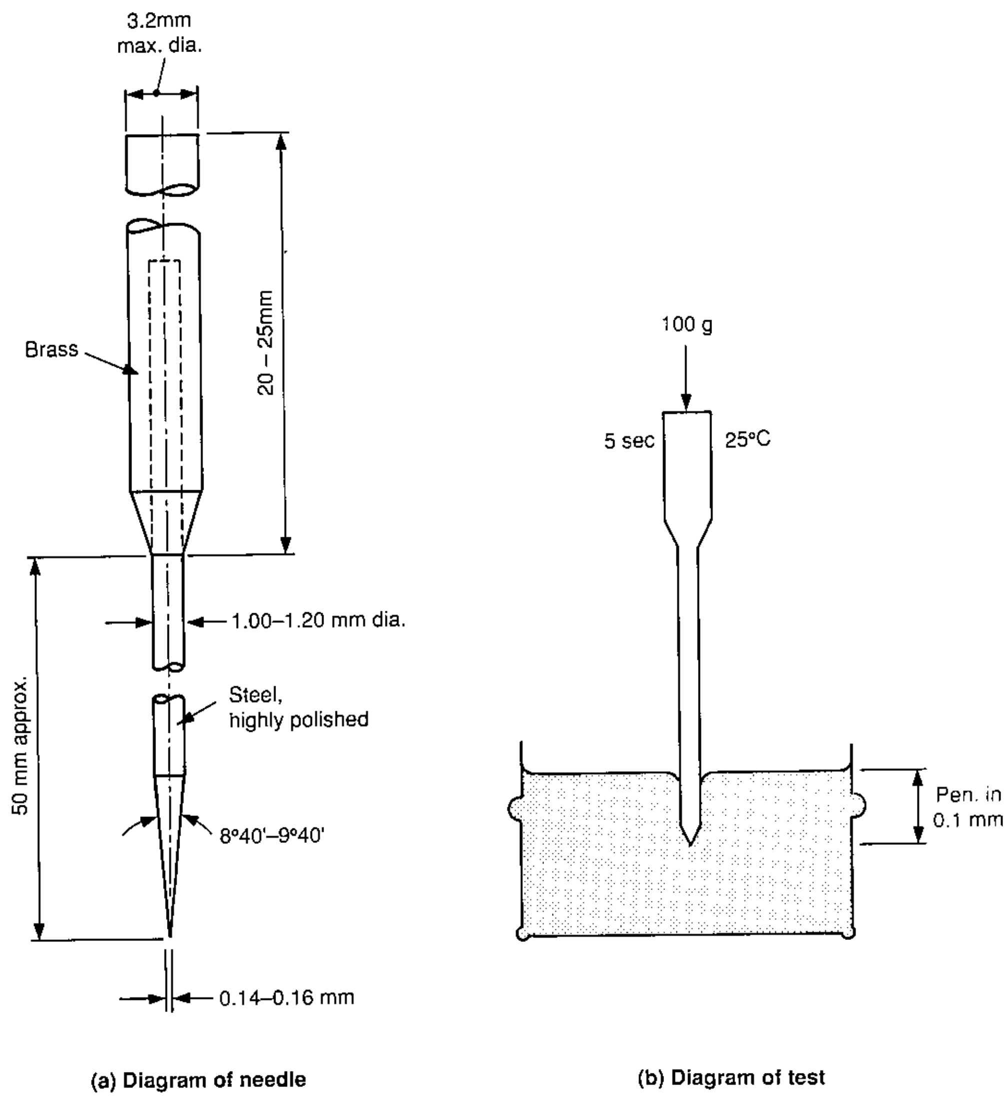


Fig.5.5 *Apparatus for the bitumen Penetration Test.*

known as blown bitumens or oxidised bitumens. They find various uses in industry, for instance, in the manufacture of roofing felts and tiles. Attempts have been made to use such blown bitumens in road asphalts but they have not proved successful, partly because of the elevated mixing temperatures that are necessary. Bitumens from Middle East crude oils are lightly blown. This is to adjust their consistency and to give them the viscosity - temperature characteristics which have been found best for use in road making.

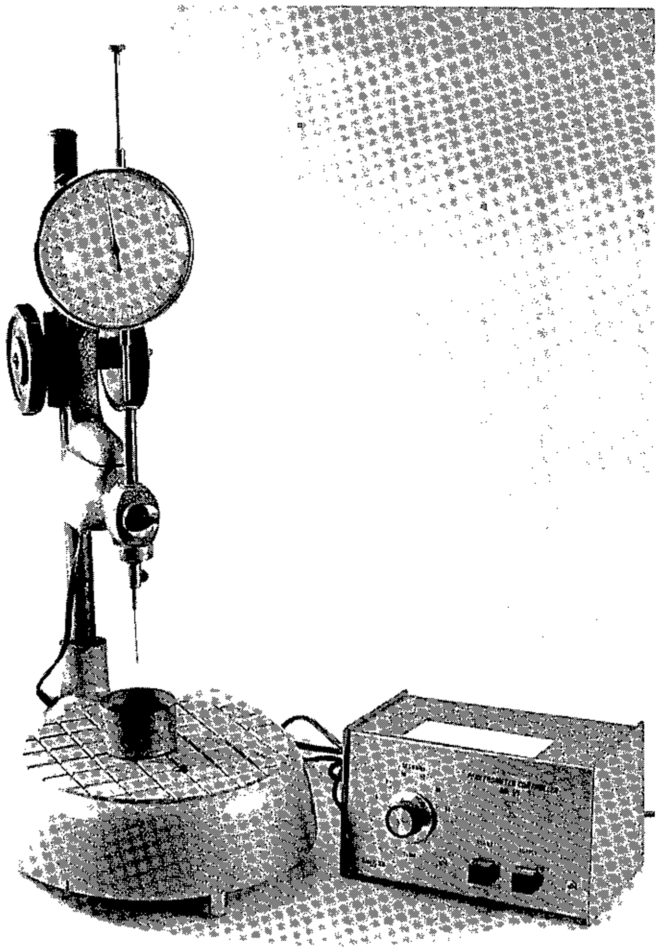
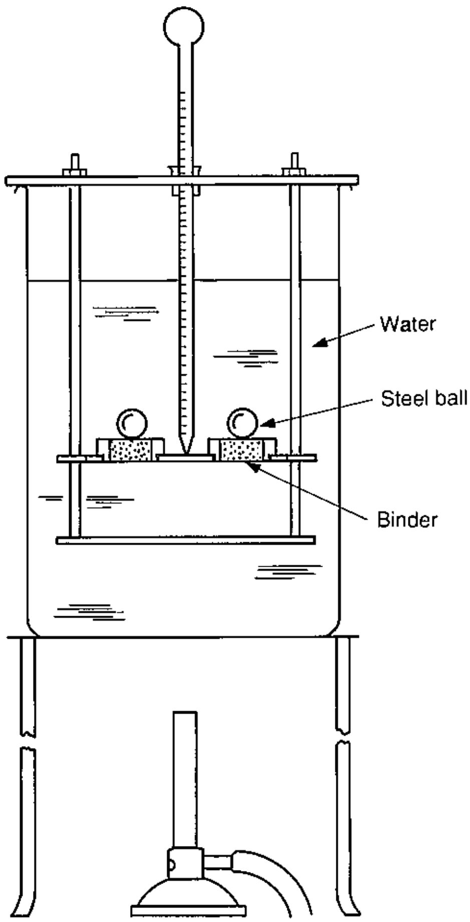


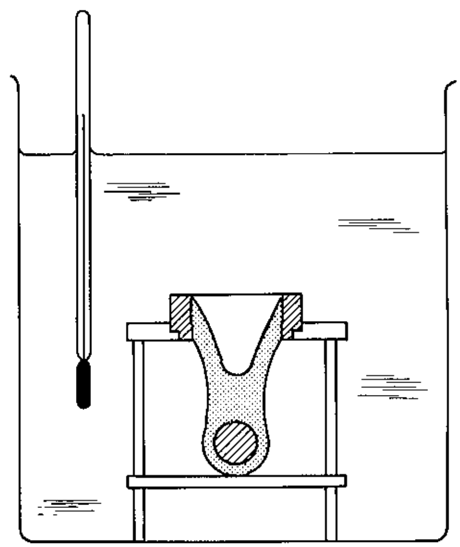
Fig.5.6 A bitumen penetrometer.

The Standards covering refined bitumens include other tests. One, solubility in trichlorethylene, gives an indication of the quality of the bitumen and is normally necessary only in evaluating bitumens from unfamiliar sources. Another is the Loss on Heating Test (BS 2000, IP 45.58). The purpose of this test is to determine how well a refined bitumen will respond to the prolonged high temperatures at which it may be stored before being used. The test requires, for example, that after heating for 5 hours at 163°C, a nominal 100 pen. bitumen shall lose no more than 0.5 per cent by mass and shall not suffer a loss in penetration of more than 20 per cent.

All refined bitumens are likely to suffer some hardening during mixing with hot aggregate and in prolonged storage of the mixture before laying. A modern trend, to store hot asphalt mixtures in insulated containers for hours and even days before laying them on the road, has increased anxiety on this score. If overheating is suspected, the bitumen can be recovered from the asphalt mixture by solvent extraction using methylene chloride (IP 105.1) and the recovered bitumen tested to determine the extent of the hardening. A reduction of more than 35 per cent in the Penetration of the bitumen would be grounds for suspicion that the mixture had been overheated.



(a) Diagram of apparatus at beginning of test



(b) Diagram showing end of test

Fig.5.7 Apparatus for the bitumen Softening Point Test.

The recovery of bitumens from asphalts that have been laid on the road for more than a few weeks may be misleading for this purpose because of possible contamination of the bitumen with oil drippings and the start of natural weathering of the bitumen.

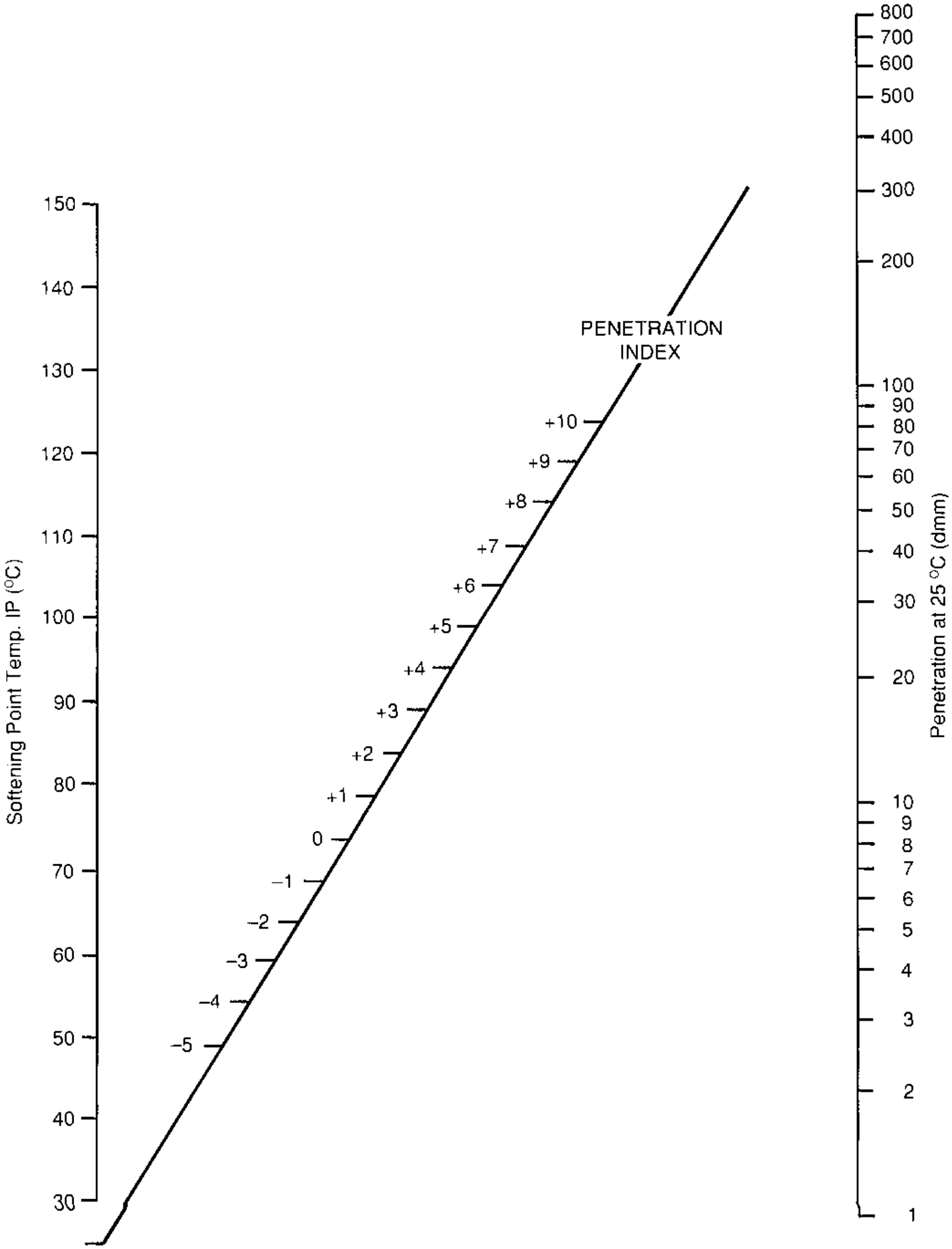


Fig.5.8 Nomograph for the Penetration Index of bitumen.

5.3.2 Cutback bitumens

BS 3690 (1989).	Bitumens for building and civil engineering. Part 1. Bitumens for roads and other paved areas.
ASTM D2026, D2027, D2028.	Specifications for cutback asphalt (slow, medium and rapid curing).

The British Standard is concerned only with cutback bitumens for use in surface dressing i.e. applied hot to the road surface before an application of chippings. There are three viscosity grades, 50 sec, 100 sec and 200 sec. measured at 40°C using the Standard Tar Viscometer with a 10 millimetre orifice as shown in Fig. 5.9. This is an efflux viscometer, and the viscosity of the cutback bitumen is defined by measuring the time in seconds for 50 millilitres of the binder to flow through the standard orifice. The test method is described in BS 2000 (1988), Part 72 (IP 72.58). The specification also defines the quantity and volatility of the distillate and the Penetration of the residue after distillation. These tests are done as a feature of compliance testing on samples taken from initial deliveries and at intervals from subsequent deliveries.

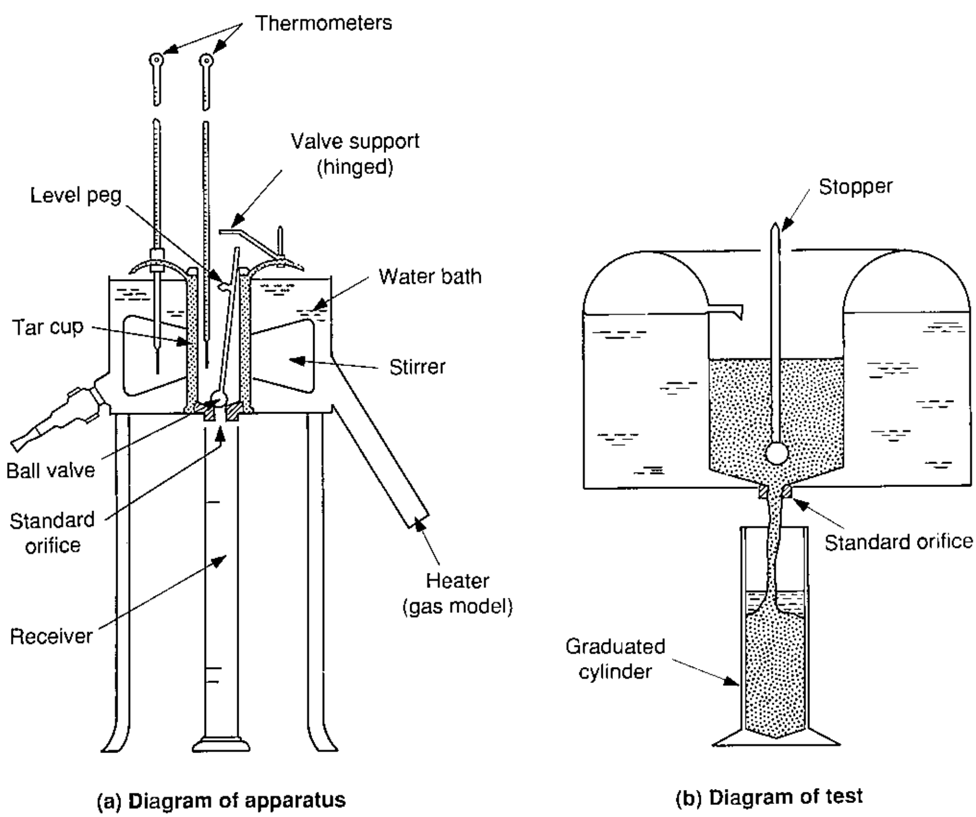


Fig.5.9 Standard tar viscometer.

The American Standards offer a wider range, both in viscosity and in volatility characteristics. The slow-curing cutbacks are intended for soil stabilisation and in the manufacture of asphalts which are intended for storage before use e.g. for patching. The medium and rapid curing cutbacks are intended mainly for surface treatments. The viscosity ranges include very fluid cutbacks which can be applied cold and which find their main use in priming gravel and stone bases prior to constructing a bituminous surface. The full range of specified cutbacks is shown below;

	Designation	*Viscosity at 60°C (Saybolt Furol) (Sec.)*				
ASTM D2026	Slow curing		SC70	SC250	SC800	SC3000
ASTM D2027	Medium curing	MC30	MC70	MC250	MC800	MC3000
ASTM D2028	Rapid curing		RC70	RC250	RC800	RC3000

The Saybolt-Furol viscometer is illustrated in Fig. 5.10. The method of test is described in ASTM D88 and E102 and in the Institute of Petroleum Handbook. Not all of the above grades are readily available, except by special order, and there is a trend to reduce the ranges specified. It may be necessary to prepare cutbacks on the spot e.g. for priming or for preparing patching material, and guidance for this is given by Hitch and Stewart (1987). Typical blends as described by Hitch and Stewart are given in Table 5.2. In preparing cutbacks in the field, great care is necessary to avoid the risk of fire.

Cutback bitumens are not suitable for making dense asphalts. Apart from the risk of fire at high mixing temperatures, the volatile components are not able to evaporate and the asphalts remain soft and incapable of carrying traffic for a considerable period. The exceptions are with bitumen-stabilised soils and with road mixes i.e. mixtures of aggregate and bitumen made by mix-in-place methods. With these, it is usual to leave the mixtures uncompacted for a period to allow the volatiles to evaporate before final compaction.

TABLE 5.2
DILUENT BLENDING RANGE FOR CUTBACK BITUMENS (EXPERIMENTAL DATA)

50 pen bitumen, nominal (61 pen actual)			*100 pen bitumen, nominal (90 pen actual)*			*100 pen bitumen, nominal (90 pen actual)*		
Cutback	Acceptable range of 3:1 kerosene/diesel (per cent by volume)		Cutback	Acceptable range of 3:1 kerosene/diesel (per cent by volume)		Cutback	Acceptable range of diesel (per cent by volume)	
MC	*Lower limit*	*Upper limit*	MC	*Lower limit*	*Upper limit*	SC	*Lower limit*	*Upper limit*
3000	14	18	3000	12	14	3000	14	17
800	21	24	800	17	21	800	20	24
250	27	32	250	24	30	250	27	33
70	37	45	70	35	44			

Outlet tube	
Int. diam.	0.315 cm
Ext. diam. (at end)	1.43 cm
Length	0.225 cm
Oil cup	
Int. diam.	2.975 cm
Height	12.5 cm

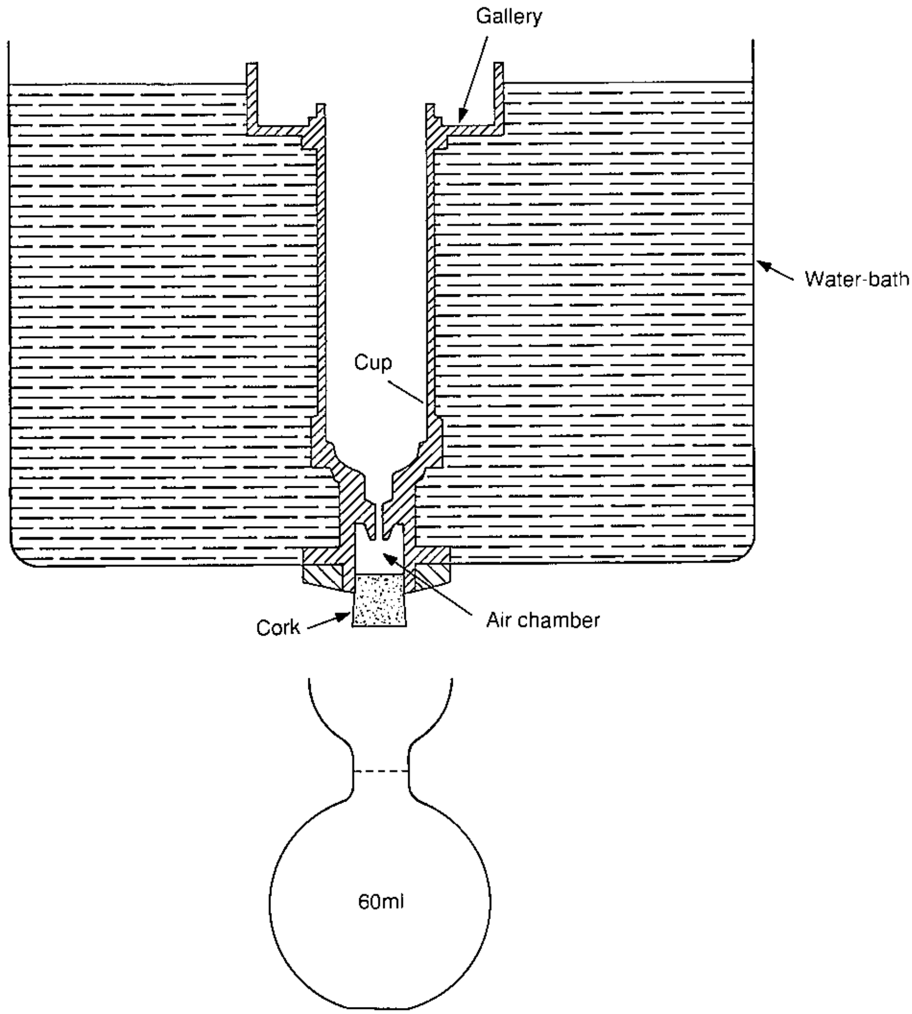


Fig.5.10 Saybolt-Furol viscometer.

5.3.3 Bitumen emulsions

BS 434 (1984).	Bitumen road emulsions (anionic and cationic)
ASTM D977.	Standard specification for emulsified asphalt
ASTM D2397.	Specification for cationic emulsified asphalt

Bitumen emulsions find their main use in surface dressing, including the special use in slurry seals. They are also used in soil stabilisation, for making patching materials, and as a tack coat to improve adhesion between asphalt surfacings and the substrate on which they are laid. They are not effective in priming newly laid gravel and crushed stone bases. Their chief advantage is that they can be used without heating, though in some countries it is possible to obtain viscous emulsions containing less than 30 per cent of water which need to be heated slightly to make them sufficiently fluid to be sprayed on the road.

BS 434 specifies eight grades of emulsion:

Anionic	*Cationic*
A1 Labile (i.e. rapid breaking)	K1 Rapid breaking
A2 Semi-stable	K2 Medium breaking
A3 Stable	K3 Slow breaking
A4 Slow breaking	
A5 Rapid (for slurry seal)	

Minimum bitumen contents are specified, and acceptance tests include viscosity, bitumen content and consistency of bitumen. Bitumen emulsions are usually made with a refined bitumen of 100 pen. grade.

The ASTM specifications cover similar ranges of cationic and anionic emulsions. Despite their obvious advantages, bitumen emulsions are, as yet, available in very few developing countries. In countries where there are major depots handling local or imported bitumen, it is clearly worthwhile to consider the local manufacture of bitumen emulsions together with the training of local staff in their use.

For the standard tests on bitumens, careful investigations have been made into their repeatibility and reproducibility. Confidence limits are quoted in the Standards and, provided that the test procedures are rigorously followed, they are quite suitable for use in specifying bitumens for contractual purposes.

5.4 Modified bitumens

For many years, attempts have been made to enhance the performance of bituminous road materials by modifying the bitumens with various additives. Amongst the most persistent has been the incorporation of rubber, either in the bitumen or with asphalts during the mixing of the bitumen with the hot aggregate. Some forms of rubber can be easily dispersed in bitumen and have a marked effect on its elastic properties. When used in asphalts, it was hoped that rubber would improve the resistance of the asphalt to cracking, particularly to the reflection cracking which occurs over joints in a concrete substrate, and also that it might enhance the overall durability of the asphalt. Some gain has been found in reducing, but not eliminating, reflection cracking in asphalts but no gain in durability has been observed, possibly because the elastic properties imparted to the binder prejudice the thorough compaction of the asphalt which is needed for good durability.

In surface dressing, some noticeable practical value has been demonstrated. Rubber in the bitumen appears to inhibit any tendency for the binder to flush up over the chippings in hot weather and also to assist in holding the chippings in place under traffic. Both effects would be expected from the reduced temperature susceptibility of the binder. The elastic properties persist in the binder as it is heated to spraying temperatures. This has a curious effect when using the whirling-spray jets that are widely used on bitumen sprayers in the United Kingdom. These jets produce a hollow conical spray of bitumen, their particular virtue being that the cones of spray from adjacent jets interpenetrate to produce an even spread of bitumen on the road surface. When using these jets to spray rubber-bitumens, the elastic properties of the binder constrain the shape of the cone so that it becomes dome shaped, as shown in Fig. 5.11, and the spread of the binder on the road can become uneven. To control this effect, the amount of added rubber must not exceed one half per cent. The effect is much less when using slotted-fan jets. With these, bitumens containing up to two per cent of rubber can be sprayed without difficulty. Rubber-bitumens for surface dressing may be commercially available in some countries. They can be made in sprayers that are equipped with circulating pumps. The rubber should never be incorporated as latex since the water it contains will produce disastrous foaming. Slightly vulcanised natural rubber is available as crumbs and, in this form, can be added to the hot bitumen and dissolved by circulating the binder for about 30 minutes. Tyre-tread rubber may also be used for this purpose. The buffings

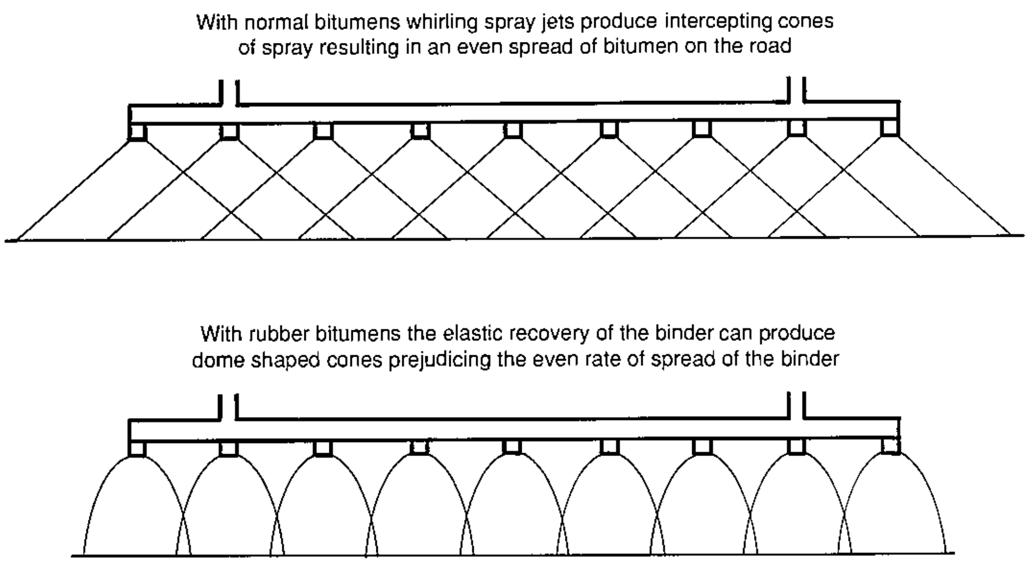
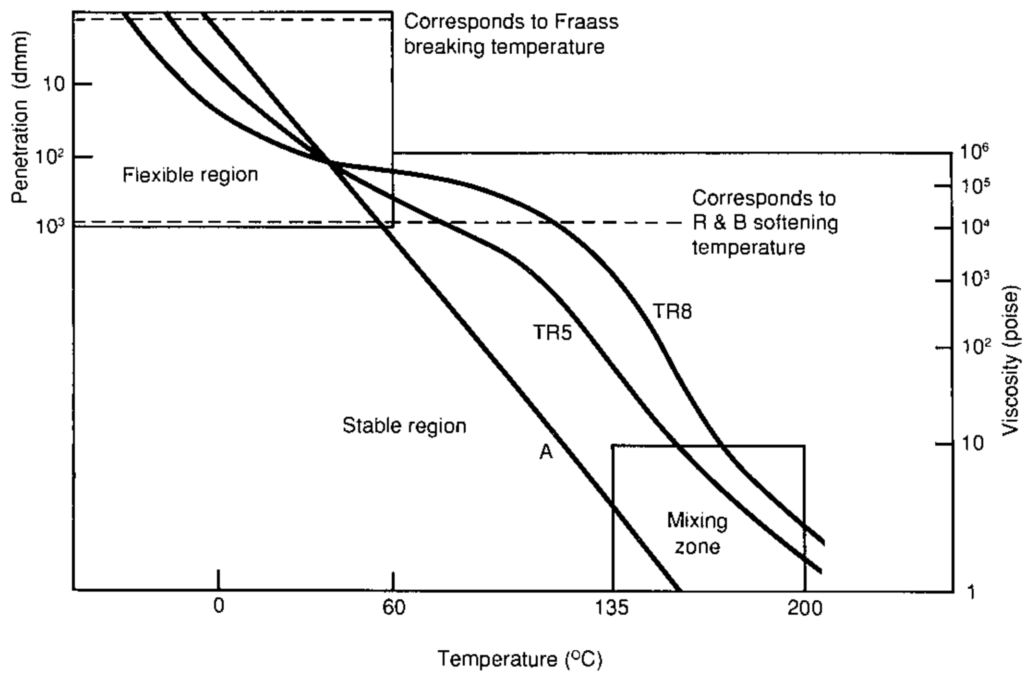


Fig.5.11 Spraying rubber-modified bitumen.



A = 80/100 pen bitumen
TR5 = 80/100 pen bitumen containing 5 per cent mass Cariflex TR–1101
TR8 = 280/320 pen bitumen containing 8 per cent mass Cariflex TR–1101

Fig.5.12 Bitumen test chart showing the improved temperature characteristics of polymer modified bitumen.

from tyres are dispersed in hot bitumen and the concentrated mixture of rubber in bitumen is supplied ready to be reheated and incorporated in the binder.

Over the last ten years there have been developments in the use of polymers incorporated in bitumen which modify their temperature susceptibility and are also claimed to improve their adhesive qualities, thus enhancing the properties of asphalts without introducing difficulties in mixing and laying. Indeed, there are claims that they can make it easier to compact the asphalts to the high densities needed for good durability. Fig. 5.12, taken from a paper by Hoban (1990), illustrates how the slope of the viscosity-temperature curve is flattened over the normal range of road temperatures, thus improving stiffness and resistance to deformation at high road temperatures and reducing brittleness at low road temperatures. Investigations in the laboratory have indicated that both the stiffness and the resistance to deformation of asphalts can be increased, effects which are important in the construction and maintenance of heavily trafficked roads, particularly in hot climates. Full scale road trails are in progress to establish whether the value of these benefits is sufficient to justify their extra cost.

Full scale trials on the road have been, and will remain, a necessary feature in extending the practical use of bituminous materials. If you are offered a material or a process which appears to have advantages in your environment, by all means try it out on the road, but always lay a control length alongside, using the best of your current practice so that you are able to assess the merits and possible demerits of the innovation. Indeed, it may well be desirable to extend the usefulness of the comparison by laying a range of compositions with both the new material and the old, for instance a range of bitumen contents with each.

5.5 Natural asphalts

Deposits of natural asphalt occur in many parts of the world, usually, but not invariably, near present day oil fields. They consist of soils or rocks which were once impregnated with crude oil and from which the lighter fractions of the oil have evaporated by geological processes. Their use by man goes back for many years. Reputedly, Noah used natural asphalt to caulk the seams of the ark, a clear indication that natural asphalts occurring in the Middle East were in regular use in boat building since early times. Sir Walter Raleigh used the asphalt from the Trinidad Lake for the same purpose, commenting in his diary in 1595 that he found it "to be most excellent good and melteth not with the Sun as the pitch of Norway and therefore for shippes trading the South parts very profitable". After natural asphalt was used to pave roads in Nebuchadnezzar's Babylon, there is no record of its use on roads until the middle 1800's when rock asphalt from Val de Travers and Neuchatel was first used to pave city streets in Europe and, about the same time, the 'tar sands' from Southern USA were being put to the same use.

Quite large deposits of natural asphalt occur in the southern states of the USA, in Indonesia on the island of Buton, in Nigeria, in Venezuela and, of course, in the famous asphalt lake in Trinidad. There are large variations in the deposits, both in the nature of the mineral content and in the hardness and quantity of bitumen. The Trinidad Lake is probably unique in this respect in that the asphalt is remarkably uniform in its composition. It consists of a mixture of sand, clay, bitumen and water. The deposit is about 50 hectares in area and occupies a volcanic basin at least 100 metres deep. An interesting review of the history and properties of Trinidad Lake Asphalt has been prepared by Attwooll and Broome (1962). The material in this deposit is in constant circulation under gaseous pressures from below and it is this circulation which is responsible for its uniform composition. Long-dead trees occasionally rise above the surface, collapse, and disappear again into the mass. As dug from the lake, the ingredients of the asphalt are present in the following proportions by weight:

Bitumen	- 39.3 per cent
Mineral and vegetable matter	- 27.2 per cent
Water and other volatiles	- 32.5 per cent

The bitumen is very hard, having a softening point in the region of 95°C. The asphalt is refined to remove the volatile materials and the vegetable and organic matter. It has been marketed for many years as Trinidad Epuré. For use in making road asphalts, it is blended with non-volatile flux oil and/or soft refined bitumen. It makes durable asphalts with the property of weathering slowly on the surface, producing a sandpaper texture which is of value when resistance to skidding in the wet is important. Its use has declined in recent years because of difficulty in handling the bitumen-mineral mixture with the binder pumping systems of modern asphalt plants.

Attempts to extract the bitumen from Trinidad Lake Asphalt and from other natural asphalts have so far proved unprofitable, but some of the other natural asphalts can be put to good use on minor roads. The Butas asphalt in Indonesia is used for this purpose even though the deposit is quite variable, both in the mineral content and in the hardness and quantity of bitumen it contains. The asphalt is dug out by hand from selected areas of the deposit and shipped by sea to other Indonesian islands where it is used with very crude equipment to build and resurface minor rural roads. The asphalt is heated, often in a split oil drum over an open fire, and is spread over a dry stone base to make a form of grouted construction which produces a tough, though rather uneven, all-weather road. The workmen appear to enjoy some local status as road builders and are keen to display their skills, another example of human resources which should adapt well to the skills required in more sophisticated road making methods.

5.6 Road tar

Crude tar is a by-product from the distillation of coal to produce coke. Formerly, it was also readily available from the processing of coal to produce coal gas but the easy availability of natural gas from oil fields has, for the time being at least, almost eliminated the demand for gas from coal. Nowadays, most crude tar is processed to extract the valuable chemicals it contains and it is unlikely that refined road tar will be offered for use in the tropics, except possibly in Zimbabwe, where refined tar may be available from the processing of coal from Wankie.

Refined road tar is a form of cutback. The crude tar is distilled to produce a soft pitch which is then fluxed with lighter tar oils to produce a material with appropriate viscosity and distillation characteristics. It has distinctly different properties from refined bitumen. Its consistency is much more affected by temperature changes i.e. as it is heated, it becomes fluid at lower temperatures than does bitumen and it hardens more rapidly as the temperature falls. Chemically, it is less stable than bitumen and is much more susceptible to hardening by the action of weather, even in temperate climates. On the positive side, it is much less soluble in fuel oils than is bitumen and it finds use in making surfacings for areas on airfields where jet-fuel may be spilled. Its adhesive qualities are superior to those of bitumen, a property that is occasionally used to improve the adhesive qualities of cutback bitumen by using creosote, a coal tar product, as a component in the cutting oils.

Road tars are used most successfully in surface dressings where their good adhesive qualities are an advantage and where their propensity to weather rapidly is mitigated by the relatively large thickness of binder film used in this process.

5.7 Adhesion

Bitumen is a good adhesive in dry conditions but there is a potential problem when wet weather occurs during the construction season because water wets stone more readily than do oil products. Fortunately there are remedies available for all the situations where water may interfere with good and persistent adhesion between bitumen and stone.

There are two requirements for good initial adhesion. The stone must be dry, with a surface free of loose dust, and the bitumen must be in a fluid condition i.e. with a viscosity of 4 poise or less (see Fig. 5.1). Thereafter, as the bitumen hardens by cooling or by the evaporation of volatile oils, it becomes more viscous. At road temperatures, it is sufficiently stiff to resist any displacement by water. Normally, if these two requirements are met, there will be no difficulty in obtaining persistent adhesion, even with siliceous rocks such as granites and quartzites. There are three particular circumstances when special precautions are necessary. One is when rain falls on a newly laid bitumen surface dressing. Another is with asphalts and bitumen macadams that are permeable to water and are made with siliceous rocks. The third lies in establishing adequate adhesion between asphalts and the substrates on which they are laid.

When surface dressings are used in renewing the surface of existing roads, the roads normally have to be kept open to traffic. Successive strips of dressing are laid along the road whilst allowing traffic to pass on one side. There are usually great pressures to open the full width of the pavement to traffic as soon as work is completed. By this time the bitumen may not have hardened completely and the surface dressings may be damaged by fast moving traffic. If, in the interim, rain has fallen, the consequences can be disastrous. Water will first displace some of the bitumen between the chippings and the road. Chippings will then be dislodged by traffic and the bitumen film will be exposed to be picked up on passing tyres, flinging both bitumen and loose chippings in all directions.

There are reliable ways of preventing such disasters. Some of them lie in the normal precautions of good surface dressing practice and are dealt with later in this book. Others lie in the magic of adhesion agents. Good practice is sufficient defence when using bitumen emulsions, anionic with calcareous rocks, cationic with others. The emulsifying agents used in these promote active adhesion between bitumen and aggregate which is sufficiently strong to resist the displacing effect of water. In climates where there is a risk of rain falling on newly laid surface dressings made with hot bitumen, similar agents can be used to prevent water from displacing the bitumen. This risk is only temporary. Once the bitumen has hardened in place in the compacted surface dressing, its high viscosity is sufficient to resist displacement by water. The last 30 years has seen many commercial adhesion agents come and go. Unmerited claims have often been made, both about the circumstances in which they are necessary and about their efficacy. Now the situation has clarified. Particularly effective adhesion agents have emerged and are widely available. These include long chain amines of high molecular weight and are similar to, if not identical with, the compounds used in the preparation of cationic bitumen emulsions.

Many different tests have been promoted for examining the adhesion between bitumen and rock. One of the most popular was the Bottle Stripping Test in which samples of stone, coated with bitumen, were immersed in distilled water for two days and the amount of stripping of bitumen from the chippings was then assessed by eye. It was difficult to quantify the judgements involved in such an assessment and, more seriously, the test did not correlate well with road behaviour, tending to overrate the value of some adhesion agents. There are still doubts about the significance of laboratory tests for adhesion. The most successful tests have been those which most closely simulate the conditions under which breakdown of adhesion occurs on the road. Thus, in evaluating the value of adhesion agents in preventing early damage to surface dressings, the most suitable test appears to be the Immersion Tray Test. This test does not appear in National Standards. A detailed description of the test has been published in the Shell Bitumen Handbook (Whiteoak (1990)) and it is briefly described below.

A tin lid, 130-140 millimetres in diameter, is covered with 15-20 grammes of bitumen to give a film of bitumen about 1.5 millimetres thick. When the bitumen has cooled to the test temperature, it is immersed in distilled water, also at the test temperature. The test temperature should approximate to the road surface temperature expected during the surface dressing work, therefore the test should not be done in an air-conditioned laboratory. Six chippings, of 14 millimetres nominal size, are lightly pressed through the water into the surface of the bitumen. The chippings are left in position for 10 minutes. They are then removed and the percentage of bitumen retained on the face of the chippings is assessed visually. The test is repeated three or four times after which it is usually clearly evident whether the chippings can establish good adhesion with the binder in the presence of water.

The usual reason for evaluating adhesion agents is to determine the amount of agent which should be incorporated in the bitumen. This will usually be between 0.5 and 2.5 per cent by weight. Mixtures with a range of concentrations of agent can be tested and the results compared with those obtained with untreated bitumen in order to indicate the effectiveness of an agent and the dosage required. Because these agents degrade with prolonged heating, they should be incorporated in the bitumen shortly before the binder is sprayed on the road.

Adhesion agents are not needed when surface dressing in dry climates nor in the dry season when there is no likelihood of rain falling within 24 hours of completing the surface dressing.

The second form of failure of adhesion can occur in moist climates with bitumen macadams and asphalts that are permeable to some degree. Siliceous aggregates are particularly vulnerable and it is quite common to find the bitumen entirely stripped from the stone in such mixtures after several years on the road. Strangely, such stripped mixtures often remain in place, apparently continuing to give good service. This happens when the pavement itself is quite strong and not subject to large transient deformations under traffic. However, in areas where the pavement is weak, potholes will develop rapidly. One of the early indications of such stripping is often the appearance of bubbles of bitumen on the road surface as the stripped bitumen is brought to the surface by the expansion of water vapour in the heat of the day.

Again, there is a simulative test which can evaluate the propensity of mixtures to such internal breakdown of adhesion. This is the Immersion Wheel-Tracking Test described by Matthews and Colwill (1962). This test requires special apparatus and is more a research tool than one for routine use. Since the defences against such internal loss of adhesion are quite simple, there is no need to include the test in the repertoire of the normal materials engineer. The defence against such internal stripping is to include 1 - 2 per cent by weight of Portland cement or hydrated lime in the bitumen-aggregate mixture. It is worth noting that such internal stripping never occurs in dense impermeable asphalts such as the rolled asphalt extensively used in the United Kingdom.

The third possible source of trouble lies in establishing adequate adhesion between asphalt surfacings and the material on which they are laid. Breakdown of this adhesion can occur at places where vehicle braking is heavy but it can also occur elsewhere on heavily trafficked roads. The normal precaution with surfacings less than 100 millimetres in nominal thickness is to apply a thin tack coat of bitumen emulsion to the substrate before laying the surfacing. When the substrate has a very smooth surface, this precaution may not be sufficient. Frequently, cores taken through newly laid asphalt surfacings show poor adhesion with the underlying layers. Adhesion between layers usually improves under traffic, but not always. Any weakness in this bond reduces the structural integrity of the pavement and prevents it from mustering the full flexural strength needed for good durability.

Good adhesion between layers requires more than the chemical adhesion supplied by a bitumen tack coat. Some mechanical interlock is also necessary. This is particularly so during the laying of the hot asphalt when the tack coat becomes molten and functions as a lubricant rather than as an adhesive; the heavy compaction necessary for good durability of the asphalt cannot be secured with material which is moving about on the substrate.

There are two remedies. In new construction involving the laying of two or more layers of asphalt, care should be taken to select a specification for the lower layers such that their exposed surfaces have a knobbly rather than a smooth texture. One way to do this is to use aggregate of a fairly large maximum size, up to 40 millimetres in the lower layers of asphalt, and to control the mix proportions so that this coarse aggregate is exposed near the surface. An important precaution is to minimise the time interval between laying successive courses of asphalt, avoiding, at all costs, the build up of a muddy film on the exposed surface of the lower layers.

On old roads that are very smooth, it may be desirable to produce a rough surface before laying the asphalt. One way to do this is to scrape the surface using the tines of a bucket excavator to produce grooves of about two or three millimetres depth. Alternatively, a surface dressing can be applied and the traffic allowed to run on the dressing for a year or more before applying the asphalt. An extreme example of this problem lies in the laying of asphalt on the stone sett paving which can still be found on some streets in cities. These setts are often highly polished under the traffic of many years and asphalts laid directly on them may slide disastrously.

There remains a phenomenon well recognised in practice but not fully explained. This concerns the use of priming coats of bitumen on newly laid stone or gravel bases to bind the surface and prevent ravelling before the bituminous surfacings are laid. It is desirable that the bitumen in these priming coats penetrates into the base for five millimetres or more. Bitumen emulsions are not effective as priming coats. They tend to break on the surface of the base, producing a film of bitumen which can be easily removed. A very fluid cutback bitumen should be used which can be applied without heating. If this is applied to a dry and dusty base, it too will harden on the surface. The base must be damp for the cutback to penetrate before it hardens. It seems likely that the fluid bitumen is able to flow over the wet surfaces of the stone. Indeed, it may be drawn in further by the suction forces created as the water evaporates. As the water disappears, the bitumen adheres strongly to the aggregate in the top few millimetres of the road base.

5.8 The weathering of bitumens

Natural weathering of bitumen begins as soon as it is used on the road. In temperate climates, this weathering is quite slow and has only a long-term effect on the durability of the asphalts. It can be considerably more rapid in hot climates. The higher temperatures and greater exposure to ultraviolet light in the tropics promote the oxidation of the bitumen which is the main factor responsible for the weathering of bitumen. Dickinson (1984) has described a thin-film oxidation test that can be used to evaluate the susceptibility of bitumens to such oxidation. Such testing will not feature in routine testing. It is used by bitumen suppliers in evaluating alternative processes in the refining of the bitumen. The concern of the road engineer is to employ the bitumen in such ways as to minimise weathering and its harmful effects on durability. There are two ways in which this can be done. One involves using processes which incorporate the bitumen in thicker rather than thinner films. Oxidation is a surface phenomenon and thick films weather more slowly than thin films. The other is to employ processes in which neither air nor water can circulate freely within the materials e.g. in dense mixtures which are well compacted.

Smith, Rolt and Wambura (1990) have reported a recent study of the weathering of bitumen in surfacings on roads in Kenya. In asphaltic concrete they found considerable hardening of the bitumen in the surface, the hardening reducing with depth (Fig. 5.13). This hardening was accompanied by the formation of a fine pattern of cracking in the surface, a phenomenon probably familiar to road engineers in the tropics.

Herein lies a dilemma. Increases in the bitumen content of surfacing mixtures may serve to enhance the resistance to weathering but are also likely to produce surfacings prone to deform in the wheel tracks under heavy traffic. It is desirable to use as high a bitumen content as the circumstances permit. Even so, such cracking may well appear in the surface within a few years and is likely to be accentuated in the wheel tracks. If untreated, it can lead to the penetration of water and progressive weakening of the pavement, which will become evident by more severe cracking.

A more certain remedy is to apply a surface dressing (seal and chip) as soon as such cracking becomes evident. The thick film of bitumen in such dressings is less susceptible to weathering and to cracking under traffic loading. It seals the cracks and it inhibits further weathering of the bitumen in the asphalt. A surface dressing is likely to extend the life of the surfacing for five years or more and there are many examples of such dressings saving pavements from rapid disruption.

Another possible defence may lie in the use of anti-oxidant compounds in the bitumen but, so far, compounds which have demonstrated some effect in laboratory oxidation tests have not shown to advantage in road trials.

Weathering of bitumen in road surfaces presents one more problem that is more acute in hot climates than in temperate zones. All tropical countries will benefit from a comparison in their own climatic environment of the resistance to weathering of different types of bituminous surfacing. The rapidity of surface weathering can be assessed in an approximate way by careful visual inspection, but it will be much more informative to obtain data on the hardening of the bitumen with time and at different depths from the surface. Bitumen recovery by solvent extraction has already been mentioned as a means of determining any changes in the consistency of the bitumen during the mixing and laying of asphalt. The same techniques can be used to recover and test weathered bitumens from road surfacings and bases. The standard bitumen recovery test is described in British Standard BS 598, Part 100 (1990) and in BS 2000, Part 105 (1991). It is rather a tedious test to do. The precision of the test is fairly good but very sensitive to any deviations from the prescribed test method. A more rapid test is now available using a rotary evaporator such as the Rotovap and this may well be a means of reducing operator error (Fig. 5.14). The method has not yet been standardised but descriptions have been given in the Shell Bitumen Handbook (1990) and it has been used extensively by Smith *et al* (1990).

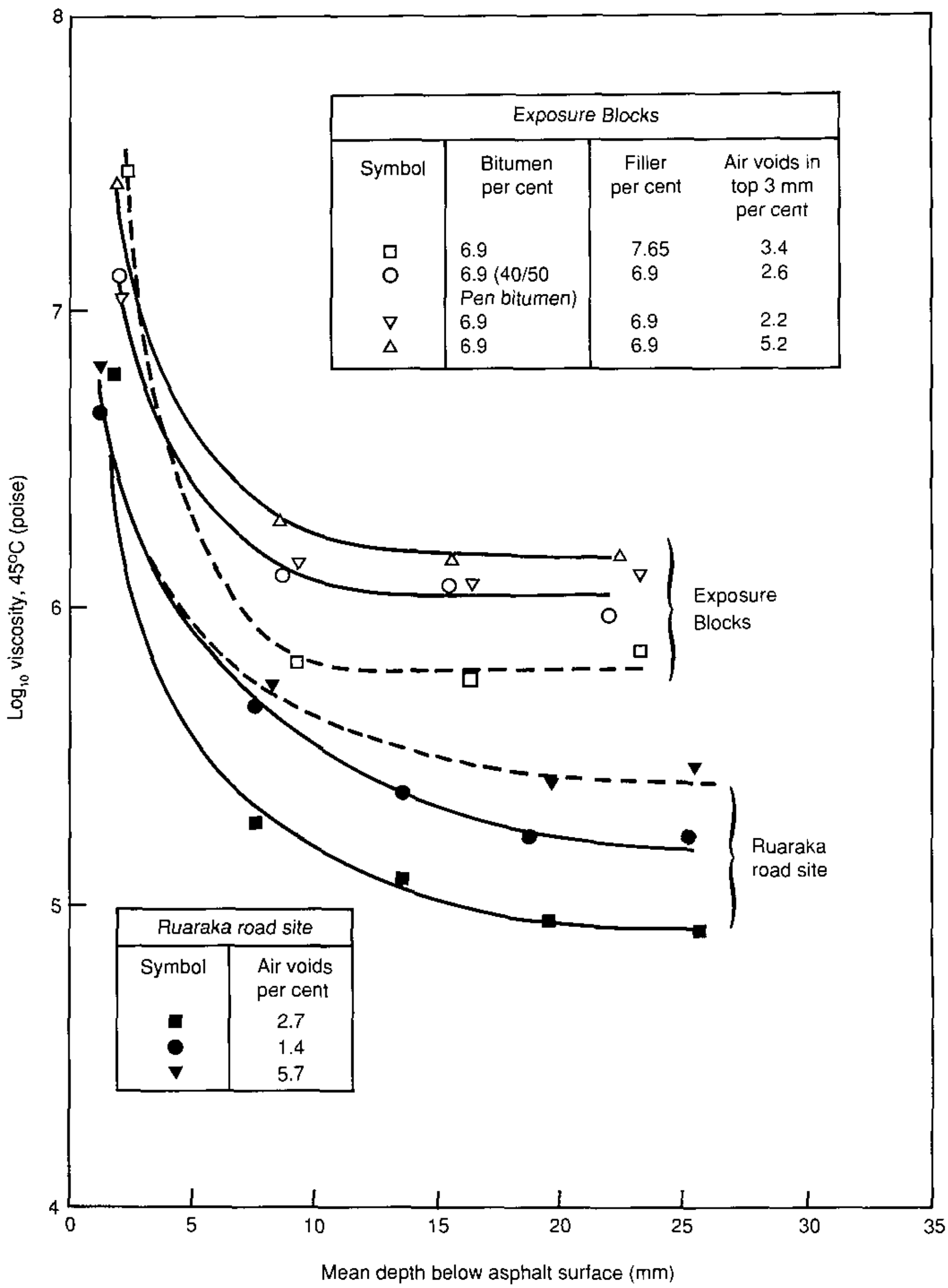


Fig.5.13 *Change in the bitumen viscosity of asphaltic concrete after 24 months exposure (in Kenya).*

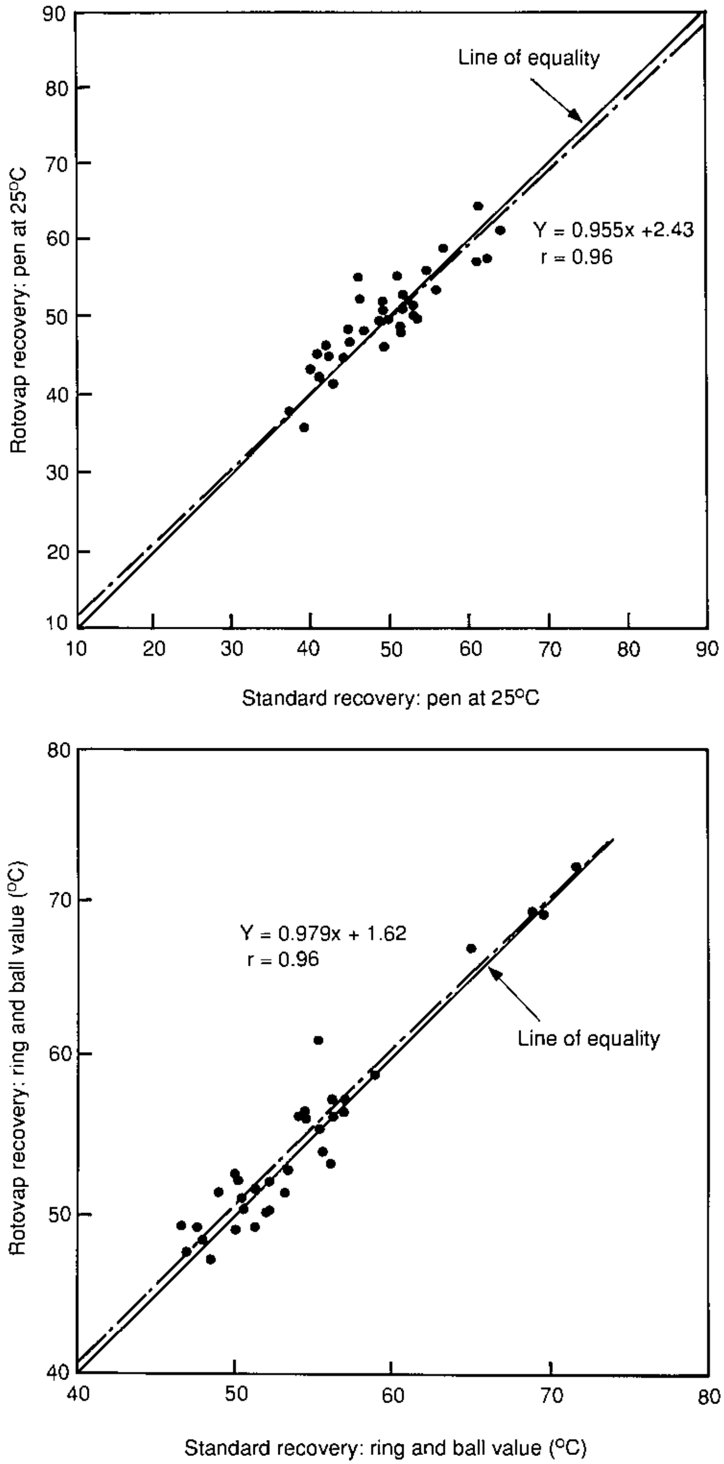


Fig.5.14 *Comparison of the properties of bitumen recovered from mixes using standard and Rotovap methods.*

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6 Bituminous premixes

6.1 Historical

Premix is a useful term covering all types of material in which aggregates are precoated with bitumen or tar before being laid on the road. There are many types of premix. Mixtures have been evolved in different countries to suit local circumstances and it would be useful to give a brief history of their development to indicate the influences that have prompted this variety.

In Europe in the second half of the nineteenth century, the need for better paving on city streets prompted the use of natural rock asphalts from southern Europe. At first, these rock asphalts, crushed to a coarse powder, were recompacted on the road with hot punners. With the advent of refined bitumen from crude oil, similar mixtures were made with fine crushed limestone. These mixtures, heated on site to a fluid condition, were spread on the road using wooden hand-floats. Hence evolved mastic asphalt, a dense, extremely strong and durable material now extensively used in civil engineering wherever a durable waterproofing layer is needed, for example, in covering flat roofs and lining reservoirs. It is used on bridge decks for this purpose and at bus stops and other places on roads where a high resistance to deformation is required. In Germany it evolved to become gussasfalt, the dense, very durable but rather smooth surfacing used on many German roads.

On other roads grouted macadam was used. This was an elaboration of Macadam's method of road building (Fig. 4.8). The upper course of broken rock, up to 50 millimetres in size, was compacted in place before being filled with fine chippings. Binder, originally crude tar from the local gas works, was then poured over the road surface, coating the stone to a depth of about 10 centimetres, and the fine chippings were then rolled in to fill the surface voids. Tar engines were used to heat the binder. These were small heaters fired by solid fuel which were equipped with small hand-operated pumps and lances to spray the binder over the road surface. This process is still in use in some countries, notably in the Indian sub-continent. It makes a tough, flexible, but rather uneven surface, prone to deform under heavy traffic but able to accommodate deformations without rupturing because of its flexibility. Unfortunately it uses rather a lot of bitumen. In an attempt to economise on bitumen, semi-grouts were introduced which used enough bitumen to penetrate for only about 5 centimetres.

Grouted macadams were soon superseded by other developments. It was realised that the grouting process could be done more quickly and cheaply by precoating the broken stone with binder before laying it on the road. Coating plants were installed in quarries and experience was rapidly gained in deriving mixtures with appropriate aggregate gradings and binder contents for different uses. Out of these came the open, medium and close textured coated macadams with compositions chosen to suit particular purposes. These are recipe mixtures i.e. their composition is specified by the grading of the aggregate, the type of binder and the binder content. It is the modern fashion to decry such recipe specifications as redolent of the cookery book but the recipes have been derived from carefully collated field experience and there can be no doubt that they work well in the environments in which they were developed.

Quite a different approach was followed in the USA. Early bituminous surfacings were made from the natural 'tar sands' of the southern States. From these, sand asphalts were evolved which used the refined bitumens as they became available. These were originally proprietary mixtures sold under trade names. Then in the early 1900's, Clifford Richardson, a materials engineer working in New York and Washington D.C. came on the scene. He did an extensive study of the sand asphalts that had been used, with varying degrees of success, to pave the streets of cities in the USA and, as a result, prepared specifications based on this experience. These were presented in his classic book, 'The Modern Asphalt Pavement', published in 1905. These mixtures provided a tough but flexible surfacing, ideal for paving the city streets and other heavily trafficked roads. They proved to be immensely durable. Indeed, some of his asphalts could still be seen in service on streets in Washington D.C. in the early 1980's. He brought these ideas to Europe and they were taken up with enthusiasm by the asphalt industry in Great Britain. They provide the source of the stone-filled sand asphalts known today as rolled asphalt.

In the meantime there had been a change in direction in the USA, prompted by the programme of Inter-State Highway construction and the need for rapidly constructed airfields in the Second World War. The demand was for asphalts of high stability that could be made from such aggregates as could be found in any locality. This prompted the development of asphalt design methods based on simple mechanical testing procedures. Three such testing procedures have survived, one developed by Bruce Marshall of the Mississippi State Highway Department (see McFadden and Ricketts (1948), one by Hubbard and Field and one by Hveem (see ASTM method D1560.81). These three tests are illustrated in Fig. 6.1. Of the three, it is the Marshall testing procedure that is now used all over the world and it is the material produced by this design procedure that is known as asphaltic concrete and sometimes as Marshall asphalt. It was probably at this time that the terms asphaltic cement and asphaltic concrete became prominent in US terminology, implying a simulation of materials bound with Portland cement. In Europe, concentrating on the programme of paving existing urban and rural roads, the emphasis on retaining the flexibility of bituminous pavements continued to influence the choice of bituminous road mixtures. Now, with the need to provide durable bituminous mixtures suitable for the very heavy traffic on motorways and other main roads, the two approaches, one by recipe from proven experience and the other by mechanical testing, are coming together in what is, as yet, rather an uneasy combination.

There are advantages and weaknesses in both approaches. Recipes derived from well-collated local experience are sure to produce good results but their use inhibits the introduction of new materials and methods. Simple mechanical testing procedures offer the prospect of employing such aggregates as are easily available in any locality but the testing regimes are only crude simulations of the action of traffic and weather on bituminous surfacings and the test results need skilled interpretation, often with modifications as experience is gained from field performance.

6.2 Physical properties of bituminous premixes

A more thorough approach, through mechanical testing, involves measurement of the particular physical properties of mixtures which are important in determining their performance under traffic on the road. These include their stiffness, resistance to deformation and strength. Research

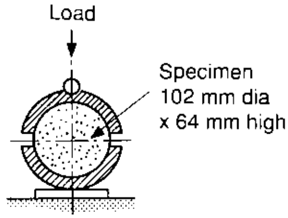
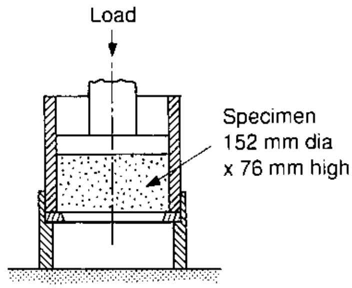
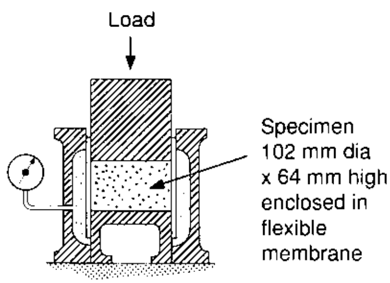
NORMAL TEST PROCEDURE	TEST AND APPARATUS	COMPACTION METHOD
Specimen compressed at 51 mm/min at 60°C Maximum load (stability), deformation at maximum load (flow value), compacted density and air voids determined	MARSHALL  	Impact by Marshall hammer (4.54 kg falling through 457 mm). Number of blows normally 50 or 75 on each face
Specimen extruded through annulus at a rate of 61 mm/min at 60°C. Maximum force determined	MODIFIED HUBBARD-FIELD  	Tamping with finishing compressive load of 45 kN for two minutes
Pressure in liquid measured at increments of load up to 26.7 kN	HVEEM COHESIOMETER  	Rotating tamper under static load of 3.4 MPa

Fig.6.1 *Three simple methods for testing the mechanical properties of bitumen-aggregate mixes.*

workers all over the world have been studying these properties in bituminous mixtures for many years. Stiffness, for instance, is an important consideration in determining the contribution of bituminous surfacings and bases to the strength of pavements. It is usually measured by bending tests on beams of the mixtures and is expressed in terms of Young's modulus of the material under dynamic loading. Resistance to deformation is important on heavily trafficked roads and assumes a particular significance in hot climates where overladen vehicles run in well-defined wheel tracks. This property is usually measured under dynamic loading, often using wheel-tracking tests employing apparatus similar to that for the Immersion Wheel-Tracking Test (Fig. 5.13). Strength i.e. resistance to rupture under strain, is not easy to evaluate since bituminous mixtures have the ability to recover after transient strains and their fatigue strength is highly dependent on the length of the rest periods between applications of load. Strength is not of great significance on strong pavements in which the transient deflections under wheel loads are small. It assumes a much greater significance on weaker pavements and under heavy vehicle loads. All three properties are considerably affected by temperature and by the hardening of the bitumen induced by weathering in hot climates. Such hardening can enhance both the stiffness and the resistance to deformation of the mixtures but is likely to reduce their fatigue strength.

The tests by which these properties are evaluated are too complex and lengthy to be used in routine testing to design mixtures for use on particular roads. Their chief value, so far, has been in characterising the properties of the different types of mixture for use in pavement design. Fig. 6.2, for example, illustrates the fatigue properties of three types of mixture (after Cooper and Pell (1974)). In evaluations of their stiffness and resistance to deformation, the ranking order

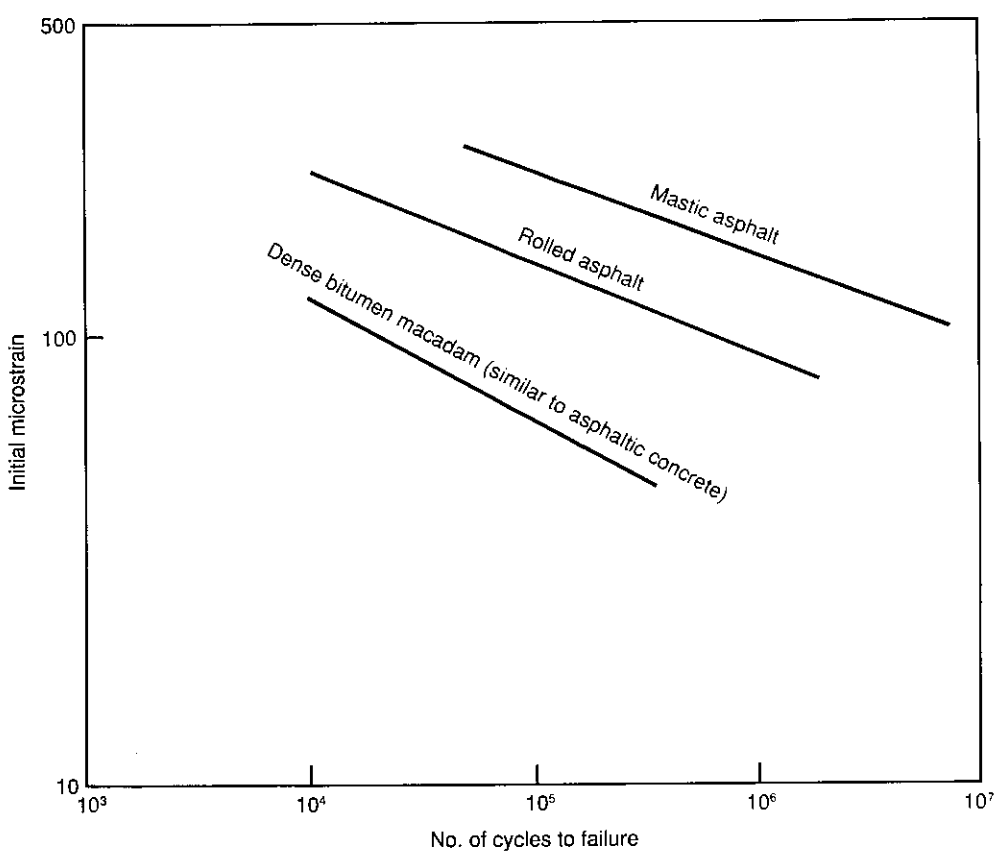


Fig.6.2 The fatigue properties of three types of bitumen-aggregate mixtures.

would be different, with mastic asphalt still the best but with asphaltic concrete mixtures usually being somewhat better than rolled asphalt mixtures.

The choice of type of bituminous mixture for use on particular roads will depend very much on local circumstances, particularly on the nature of the materials available for making the mixtures. On roads carrying heavy vehicles and on airfields, it will be desirable to aim for high stiffness and resistance to deformation. The Marshall testing regime is currently the most convenient for the routine design of premixed surfacings. It can be used in designing rolled asphalt surfacings as well as asphaltic concrete. Dense coated macadam surfacings can be used on less heavily trafficked roads but the Marshall testing regime cannot be used in designing these mixtures. Their formulations are derived from carefully collated road experience and typical examples used in Bermuda and Jamaica are described in Table 4.5.

Bituminous bases and basecourses are likely to be used increasingly on more heavily trafficked roads in Third World countries. With these there are advantages in using aggregate gradings with particles up to 50 millimetres in size but mixtures with particles over 20 millimetres in size cannot be designed by any of the simple mechanical testing procedures. The properties of bituminous bases have been extensively studied in the laboratory, in full scale trafficking trials and in road experiments. These studies have established the effects of aggregate grading and bitumen content on stiffness, fatigue strength and resistance to deformation. The results are incorporated in standard specifications in the form of recipe mixtures. Such studies are now revealing the extent to which the use of polymer modified bitumens can increase the stiffness and resistance to deformation of such mixtures (see Section 5.4). Bases made with polymer modified bitumens for use on very heavily trafficked roads are soon to be included in standard specifications and should prove of particular value in hot climates.

In the tropics it is desirable to aim for mixtures with low voids through which air and water vapour cannot circulate in order to reduce the effects of weathering on the bitumen. Thorough compaction is necessary, both to enhance stability and to eliminate interconnecting voids.

To help in choosing the most appropriate type of premixed bituminous surfacing and base for particular circumstances, the notes below describe the composition and characteristics of mastic asphalt, asphaltic concrete, rolled asphalt and bitumen macadams. References are given to the appropriate British and American Standards for these materials. As experience is gathered in other countries, it is desirable that national specifications should be established for the mixtures which prove most successful in the local environment.

6.2.1 Mastic asphalt

It is not likely that mastic asphalt will be readily available in many developing countries. Specialist skills and equipment are needed in its manufacture and laying but it is possible that, in urban areas, the value of the material in building e.g. in roofing, reservoir lining etc., may encourage its introduction and provide opportunities for use in paving bus stops, bridge decks and roads carrying very heavy loads on industrial sites. In such circumstances its rather high cost may be justified by its superior performance. Modern mastic asphalts are made with fine aggregate (maximum size 5 millimetres), usually limestone, but other aggregates may be used. Relatively hard bitumens are employed, usually between 15 and 25 pen.. Compared with other premixes, mastic asphalts have high bitumen contents, between 14 and 17 per cent by weight (i.e. between 30 and 35 per cent by volume).

These mixtures are supplied to the site in blocks which are melted in a mastic heater and are then spread in a semi-molten condition using hand floats. The German gussasfalt, a special form of mastic asphalt with a lower bitumen content, is laid by machine.

In many ways mastic asphalt is the ideal form of bituminous premix. It has a high stiffness and strength, and is highly resistant to deformation. It is extremely durable and, being dense and voidless, is less susceptible to the weathering to which bitumens are vulnerable in warmer climates. It has two disadvantages for use on roads. One is its smooth surface which is very slippery when wet. This disadvantage can be overcome by rolling precoated chippings into the surface whilst the asphalt is still warm, by surface dressing, or by the application of some other anti-skid treatment. The other disadvantage is its relatively high cost. Nevertheless, where expertise in the use of mastic asphalts is available, this high cost may be justified for the special uses listed above.

The relevant British Standards are:

BS 1446 (1973). Specification for mastic asphalt (natural rock asphalt fine aggregate) for roads and footways.

BS 1447 (1988). Specification for mastic asphalt (limestone fine aggregate) for roads, footways and pavings in building.

There is no equivalent ASTM specification.

6.2.2 Asphaltic concrete

The design of asphaltic concrete mixtures starts with the assumption that the particle size distribution of the aggregate should be such as to produce the highest possible compacted density in the aggregate fraction of the mix. For this purpose the aim is normally to use a continuously graded aggregate following a Fuller curve. The Fuller curve is derived from the formula

$$P = 100 \times (d/D)^{0.5}$$

where P is the percentage of aggregate passing sieve size d, and D is the maximum size of aggregate in the mixture. This formula was derived by Fuller and Thompson (1907) in an investigation of the relationship between the compacted density of Portland cement concrete and the grading of the aggregate used in its manufacture. Typical grading limits for aggregate in asphaltic concrete, as recommended by the Asphalt Institute, are given in Table 6.1. The bitumen used for asphaltic concrete is normally in the range 60-100 pen..

Mixtures of aggregate of the chosen grading and bitumen of the selected penetration grade are evaluated using the Marshall testing procedure. This testing procedure has been standardised both in the USA and in the United Kingdom. The relevant standards are:

TABLE 6.1
COMPOSITION OF ASPHALTIC CONCRETE (ASPHALT INSTITUTE)

Sieve size mm	Percentage by mass of total aggregate passing				
	37.5 mm	*25.0 mm*	*19.0 mm*	*12.5 mm*	*9.5 mm*
50	100	-	-	-	-
37.5	90 - 100	100	-	-	-
25.0	-	90 - 100	100	-	-
19.0	56 - 80	-	90 - 100	100	-
12.5	-	56 - 80	-	90 - 100	100
9.5	-	-	56 - 80	-	90 - 100
4.75	23 - 53	29 - 59	35 - 65	44 - 74	55 - 85
2.36	15 - 41	19 - 45	23 - 49	28 - 58	32 - 67
1.18	-	-	-	-	-
0.60	-	-	-	-	-
0.30	4 - 16	5 - 17	5 - 19	5 - 21	7 - 23
0.15	-	-	-	-	-
0.075	0 - 5	1 - 7	2 - 8	2 - 10	2 - 10
Asphalt cement, weight percent of total mixture	3 - 8	3 - 9	4 - 10	4 - 11	5 - 12

1 In considering the total grading characteristics of an asphalt paving mixture the amount passing the 2.36 mm (No 8) sieve is a significant and convenient field control point between fine and coarse aggregate. Gradings approaching the maximum amount permitted to pass the 2.36mm (No 8) sieve will result in pavement surfaces having comparatively fine texture, while gradings approaching the minimum amount passing the 2.36mm (No 8) sieve will result in surfaces with comparatively coarse texture.

2. The material passing the 0.075 mm (No 200) sieve may consist of fine particles of the aggregates or mineral filler, or both. It shall be free from organic matter and clay particles and have a plasticity index not greater than 4 when tested in accordance with Method D 423 and Method D 424.

3. The quantity of asphalt cement is given in terms of mass percent of the total mixture. The wide difference in the specific gravity of various aggregates, as well as a considerable difference in absorption, results in a comparatively wide range in the limiting amount of asphalt cement specified. The amount of asphalt required for a given mixture should be determined by appropriate laboratory testing or on the basis of past experience with similar mixtures, or by a combination of both.

ASTM D1559. Marshall testing procedure.

BS 598 (1985). Sampling and examination of bituminous mixtures for roads and other paved areas. Part 3. Methods for design and physical testing.

The two methods are similar, the British being more detailed in an attempt to improve the consistency of results obtained from this rather complex testing procedure. The form of the test is shown diagrammatically in Fig. 6.1. A range of mixtures is prepared in the laboratory and compacted by a standard method (a dropping hammer) in a cylindrical test mould 101.6 millimetres in diameter and 87.4 millimetres high. The compacted density is then measured. Thereafter, the specimen, at a test temperature of 60°C, is mounted horizontally between special jaws and a compressive load is applied at a constant rate of strain of 50 millimetres per minute. The maximum load (the Marshall Stability) and the amount of strain at maximum load (the Flow Value) are recorded. Careful temperature control is necessary at all stages since compactability, stability and flow are all influenced by changes in the viscosity of the bitumen in the mixtures.

Tests are done with mixtures containing a range of bitumen contents, triplicate tests at each bitumen content, to derive mean values of density, stability and flow. A typical set of results is illustrated in Fig. 6.3. These are used to derive an optimum bitumen content and an indication of the density to which the asphalt should be compacted on the road.

It is frequently found that adjustments have to be made to the optimum bitumen content to suit particular types of aggregate, different climatic conditions and different traffic loadings. Inexperienced engineers tend to regard the testing regime as an inviolate arbiter of design but it is a very crude simulation of the complex physical stresses that asphalts are subjected to during compaction and under subsequent traffic. In comparing mixtures with different formulations, it can give misleading results. It should be regarded as a component of a design procedure in which local experience must play a major part.

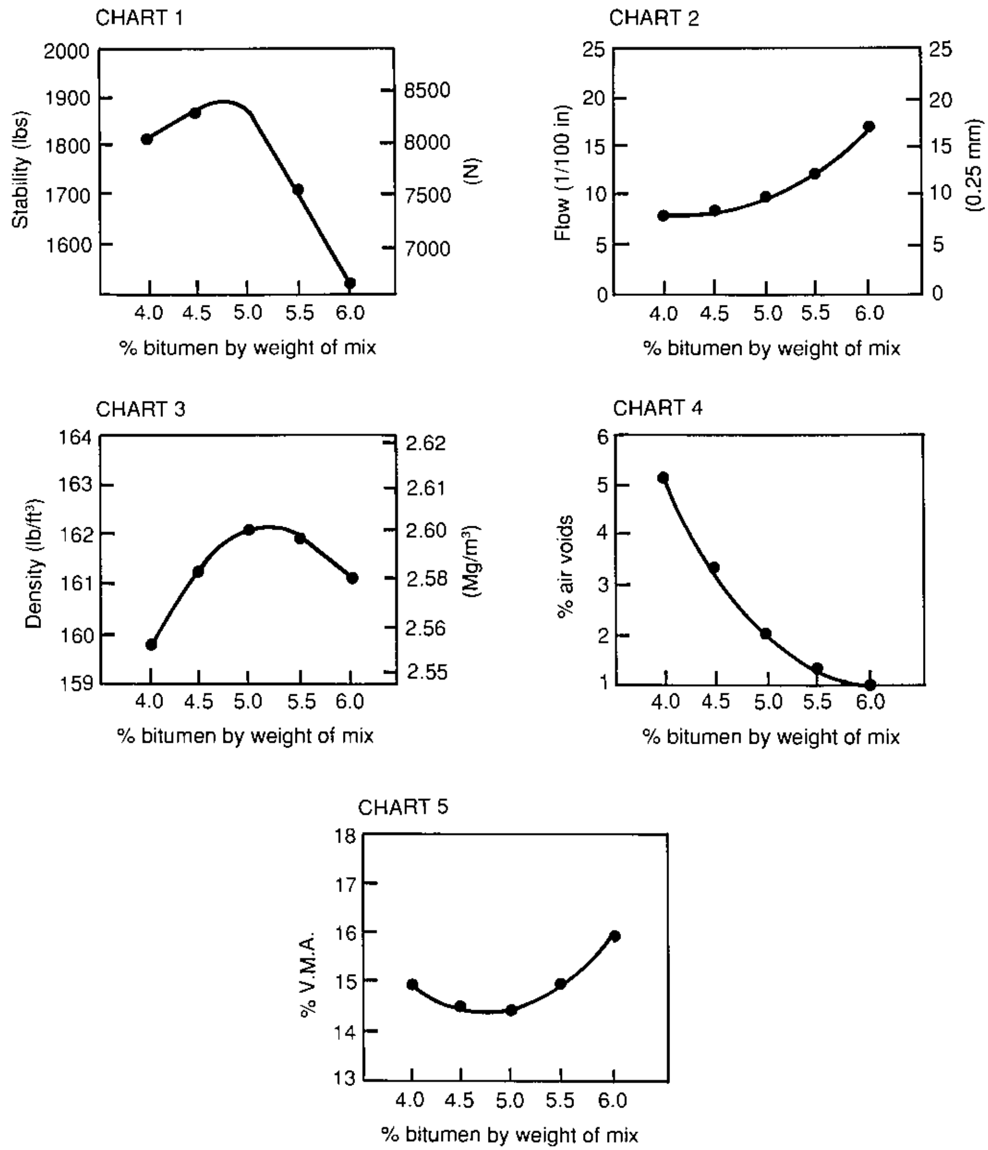
The Marshall testing procedure is also used by some engineers in the design of rolled asphalt surfacings but others continue to rely on the compositions derived from experience which are directly specified by type and grading of aggregate and by bitumen content and consistency (see Section 6.2.3).

Limits of precision for the Marshall testing procedure are not quoted in either the American or the British Standard. It must be said that, in practice, the test is often found to give very varying results. A likely explanation for this poor reproducibility can be inferred from the following extract from the description of the test method in BS 598.

"It must be emphasised that the design procedure is not simple and takes time. At least three consecutive working days are needed to carry out the procedure properly and great care and attention to detail are essential if a reliable result is to be obtained. Precise conformity to all the requirements for apparatus and procedure will give a binder content that can be used with confidence in specifications. Other equipment and the use of short cuts and variation in procedure will almost certainly give misleading results"

The test is for design purposes only. It is sometimes suggested that the test should be used in compliance testing to make sure that the asphalt supplied is consistently in accord with a specification. There is an airy logic in this suggestion but such compliance testing involves the sampling and testing of asphalt from the newly mixed material at the asphalt plant. This is a variation from the standard testing procedure which is unlikely to produce consistent and representative results.

The test should be used as part of a design procedure which defines a 'job mix' specified in terms of the type and target grading of aggregate, the grade of bitumen and the target bitumen content, together with such other controls as are necessary on the mixing and laying processes.



Optimum binder content for stability = 4.8%
Opimum binder content for density = 5.1%
Binder content at 4% air voids = 4.3%
Design binder content $= \frac{4.8 + 5.1 + 4.3}{3} = 4.7\%$

Fig.6.3 Results from the Marshall Test.

6.2.3 Rolled asphalt

Rolled asphalt surfacings were traditionally made with natural sands with varying proportions of larger crushed stone particles added according to the use intended. In the United Kingdom in recent years, quarry owners have sought to replace the natural sand with crushed stone fines in order to employ their own crushed stone products. The essential feature of rolled asphalt surfacings is that they consist of a matrix of fine aggregate and bitumen in which coarser aggregate is incorporated i.e. the aggregate is gap-graded as opposed to the continuous grading of asphaltic concrete. The matrix provides a dense, impermeable and flexible quality to the surfacing. Its stability and resistance to deformation is enhanced by using somewhat harder bitumens than are used in asphaltic concrete. The proportion of coarse aggregate affects the surface texture of the material when laid. With mixtures containing 30 per cent or less of coarse aggregate, the compacted surface is smooth and the rough nobbly surface texture, which is needed to produce an adequate resistance to skidding on high speed roads, is provided by rolling in precoated chippings to give a complete cover over the surface (Fig. 6.4). This provides an adequate texture depth and also contributes to resistance to deformation. It is this material which is used to surface motorways and other busy roads in the United Kingdom. On more lightly trafficked roads, mixtures containing 45-50 per cent of coarse aggregate are used. These have a coarse surface texture providing an adequate resistance to skidding without the use of precoated chippings. The composition, mixing and laying of rolled asphalt mixtures is prescribed in the following British Standards:

BS 594 (1985).	Hot rolled asphalt for roads and other paved areas.
Part 1.	Constituent materials and asphalt mixtures.
Part 2.	Transport, laying and compaction of rolled asphalt.

This specification includes surfacings, basecourses and bases of rolled asphalt, and the composition of these is defined in terms of grading of aggregate, hardness of bitumen and bitumen content. In this current British Standard there is an alternative method of designing rolled asphalt surfacings using the Marshall testing regime and the method of carrying out the test and reporting the results is given in BS 598 (1985), Part 3. Typical compositions for rolled asphalt surfacing mixtures are shown in Table 6.2.

These compositions have been derived from experience in the United Kingdom over the last 50 years. In countries with different climates, it will be desirable to employ the Marshall testing regime to obtain an indication of the likely suitable range of bitumen contents, refining the specifications as experience is gained in practice. Changes in the nature of the fines fraction, particularly between natural sand and crushed rock fines, can have a large influence on the required bitumen content.

It is generally recognised that these mixtures are more durable and flexible than the continuously graded asphaltic concretes. They are also less sensitive to minor variations in bitumen content and they are easier to lay and compact to their final density on the road. These are advantages that commend their use in hot climates but they can be more sensitive to deformation under heavy traffic in hot weather and careful design of the mixture is necessary to avoid this risk. In pursuing the advantages of these mixtures for use in South Africa, Marais (1974) established Marshall Test criteria to enhance their resistance to deformation. He proposed that the ratio of filler (i.e. fine aggregate passing the 75 micron sieve) to bitumen should never be less than unity. To safeguard

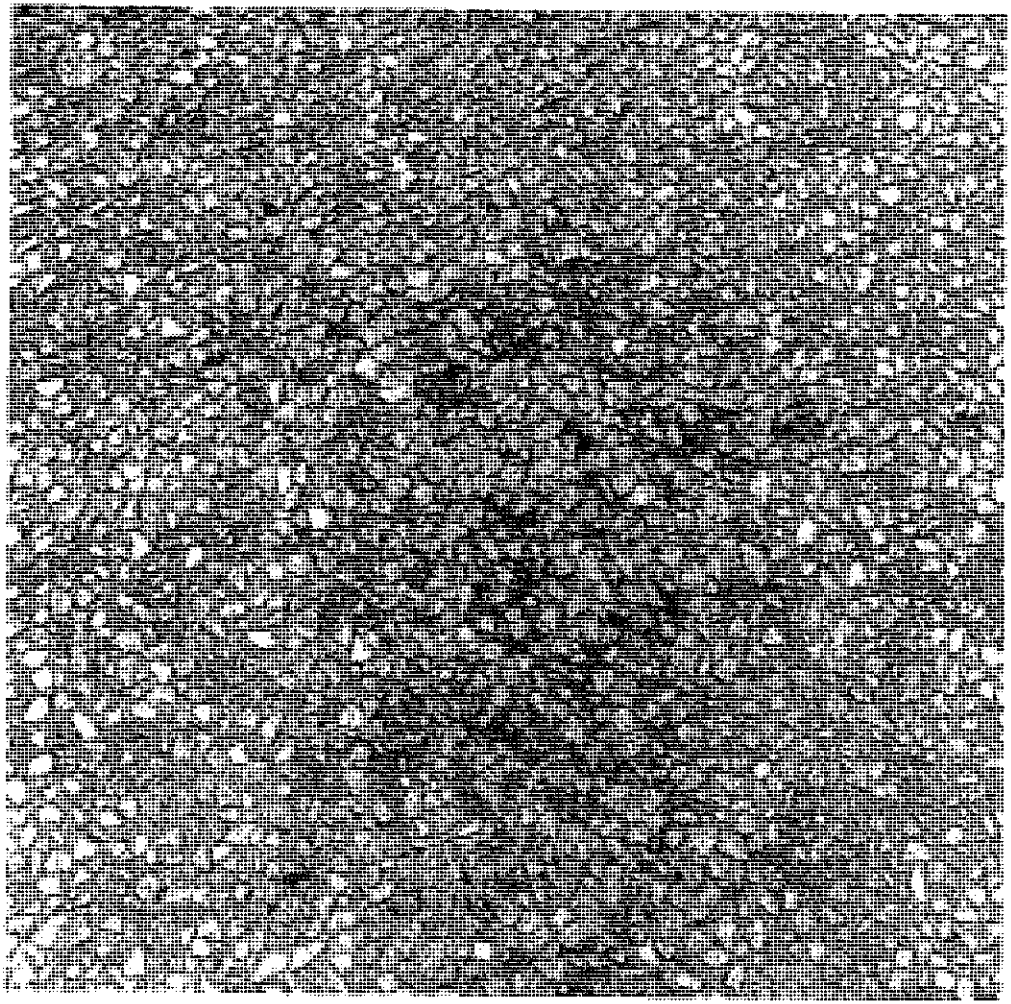


Fig.6.4 Hot rolled asphalt surface with precoated chippings.

durability, he suggested that the average film thickness of the bitumen should not be less than 6.0 microns. This average film thickness can be determined by calculating the surface area of the aggregate from its grading, using the factors recommended by the Asphalt Institute. The average film thickness in asphaltic concretes is usually considerably less than this. With the Marshall testing procedure now included in BS 594, it is possible to design mixtures for use in hot climates with high stabilities and middle range flow values. Such mixtures have been used successfully in South Africa for the last 20 years, employing mine tailings and other locally available fine aggregates. More recently, such mixtures have been brought into good use in Indonesia where their flexibility has been found to be particularly valuable in surfacing roads over the weak subgrades of unconsolidated silts in the low lying padi lands (Corne (1983) and Dardak *et al* (1992)).

TABLE 6.2
ROLLED ASPHALT WEARING COURSE MIXTURES (TAKEN FROM BS 594, 1985 PART 1)

Composition of design type F	Percentage by mass of total aggregate passing test sieve					
Column number	7	8	9	10	11	12
Designation(a)	0/3(b)	30/10	30/14	40/14	40/20	55/20
Nominal thickness of layer (mm)	25	35	40	50	50	50
BS test sieve (mm)						
28	-	-	-	-	100	100
20	-	-	100	100	95 - 100	90 - 100
14	-	100	85 - 100	90 - 100	50 - 85	35 - 80
10	-	85 - 100	60 - 90	50 - 85	-	-
6.3	100	60 - 90	-	-	-	-
2.36	95 - 100	60 - 72	60 - 72	50 - 62	50 - 62	35 - 47
0.6	80 - 100	45 - 72	45 - 72	35 - 62	35 - 62	25 - 47
0.212	25 - 70	15 - 50	15 - 50	10 - 40	10 - 40	5 - 30
0.075	13 - 17	8 - 12	8 - 12	6 - 10	6 - 10	4 - 8
Maximum percentage of aggregate passing 2.36 mm and retained on 600 μm BS test sieves	22	14	14	12	12	9
Minimum target binder content % by mass of total mixture(c)	9.0	7.0	6.5	6.3	6.3	5.3

(a) The mixture designation numbers (eg 0/3 in column 7) refer to the nominal coarse aggregate content of the mixture/ nominal size of the aggregate in the mixture, respectively.
(b) Suitable for regulating course.
(c) In areas of the country where prevailing conditions are characteristically colder and wetter than the national average the addition of a further 0.5% of binder may be beneficial to the durability of the wearing courses.

Note: These mixtures are usually associated with the use of a natural sand fine aggregate although other fine aggregates complying with the grading limits can be used.

With gap-graded rolled asphalt surfacings, it is the properties of the sand-filler-bitumen mortar which dominate the properties of the total mix. The nature of the sand is particularly important in determining the resistance to deformation of the mix. Wheel-tracking tests have demonstrated that there are large differences in both the resistance to deformation and the optimum bitumen contents of mixtures made with different sands, all conforming to the grading limits of BS 594. Brien (1978) has developed a method of using the Marshall testing procedure for evaluating the sand-filler-bitumen mortar which is particularly useful in evaluating the potential of locally available natural sands and crushed rock fines for use in rolled asphalt surfacings. Optimum bitumen contents are determined from tests on sand-filler-bitumen mixtures and the qualities of the mixtures with different sands are compared by using the Marshall Quotient i.e. the ratio of

TABLE 6.3
ROLLED ASPHALT ROADBASE AND BASECOURSE MIXTURES (TAKEN FROM BS 594, 1985)

	Percentage by mass of total aggregate passing test sieve					
Column number	1	2	3	4	5	6
Designation*	50/10†	50/14†	50/20†	60/20	60/28	60/40
Nominal thickness of layer (mm)	25 to 50	35 to 65	45 to 80	45 to 80	60 to 120	75 to 150
BS test sieve (mm)						
50	-	-	-	-	-	100
37.5	-	-	-	-	100	90 - 100
28	-	-	100	100	90 - 100	70 - 100
20	-	100	90 - 100	90 - 100	50 - 80	45 - 75
14	100	90 - 100	65 - 100	30 - 65	30 - 60	30 - 65
10	90 - 100	65 - 100	35 - 75	-	-	-
6.3	-	-	-	-	-	-
2.36	35 - 55	35 - 55	35 - 55	30 - 44	30 - 44	30 - 44
0.6	15 - 55	15 - 55	15 - 55	10 - 44	10 - 44	10 - 44
0.212	5 - 30	5 - 30	5 - 30	3 - 25	3 - 25	3 - 25
0.075	2 - 9	2 - 9	2 - 9	2 - 8	2 - 8	2 - 8
Binder content % by mass of total mixture for:						
Crushed rock or steel slag	6.5	6.5	6.5	5.7	5.7	5.7
Gravel	6.3	6.3	6.3	5.5	5.5	5.5
Blastfurnace slag. Bulk density (kg/m^3)						
1440	6.6	6.6	6.6	5.7	5.7	5.7
1360	6.7	6.7	6.7	5.9	5.9	5.9
1280	6.8	6.8	6.8	6.0	6.0	6.0
1200	6.9	6.9	6.9	6.1	6.1	6.1
1120	7.1	7.1	7.1	6.3	6.3	6.3

*The mixture designation numbers (e.g 50/10 in column 1) refer to the nominal coarse aggregate content of the mixture/nominal size of aggregate in the mixture, respectively.

†Suitable for regulating course.

Marshall Stability to Marshall Flow. Mixes using crushed rock fines generally have a higher stability than those made with natural sands but they are less workable and more difficult to compact. There are some circumstances in which it is useful to use mixtures of crushed rock fines and natural sand to produce adequate resistance to deformation combined with ease in spreading and compaction.

Rolled asphalt bases and basecourses are specified in terms of aggregate grading, hardness of bitumen and bitumen content. These are reproduced in Table 6.3. In this Table the term 'regulating course' refers to the two-course surfacing procedure commonly used in the United Kingdom in which a total thickness of about 100 millimetres is made up of a running surface, normally 35-40 millimetres thick, and a lower regulating course, one purpose of which is to remove bumps and hollows in the substrate on which the surfacing is laid. Table 6.3 illustrates how the maximum size of aggregate in asphalt bases is increased as the layer thickness increases. This economises on the use of bitumen and helps in producing stiffer mixtures. With all forms of asphalt premix, satisfactory compaction requires that the maximum size of aggregate in the mixture should never exceed half of the thickness of the layer in which the mixture is used. Laboratory studies and full scale road experiments have established that rolled asphalt bases are extremely strong, durable and superior to other forms of base (see Croney (1991)).

6.2.4 Coated macadams

These are mixtures in which the largest size of aggregate in the aggregate grading predominates. They now range from the very open-textured material known as pervious macadam and used as an anti-splash surfacing, through to dense, relatively impervious mixtures used as basecourses and roadbases. Those with open gradings are not likely to be of value in hot climates because of their permeability and the likelihood of rapid hardening of the bitumen from weathering.

In the United Kingdom coated macadams are specified in the following British Standard:

BS 4987 (1988).	Coated macadam for roads and other paved areas.
Part 1	Specifications for constituent materials and for mixtures.
Part 2	Transport, laying and compaction.

There are no equivalent ASTM specifications but the Asphalt Institute's Handbook on mix design methods contains information on these materials, as does the handbook on Hot Mix Asphalt Paving recently published by U.S. highway authorities (US Army Corps of Engineers (1991)).

The aggregate gradings of the dense coated macadam basecourses specified in BS 4987 are given in Table 6.4 together with the specified target bitumen contents. The grades of bitumen specified for these mixtures are 100 pen. and 200 pen.. These mixtures have lower bitumen contents than the high quality rolled asphalt bases and are therefore somewhat cheaper. They are generally slightly less effective in mustering the high dynamic stiffness needed on heavily trafficked roads. Even so, the dense bitumen macadam bases and basecourses find use in hot countries provided that they are surface dressed to protect them from weathering.

Pervious macadam wearing courses are not likely to be durable in hot climates but they are worth describing because of their interesting properties. They derive from the friction courses developed in the late 1950's to prevent aquaplaning on the runways of military airfields. The mixtures contain only a small proportion of fine aggregate so that when they are laid and compacted they contain 15-20 per cent of interconnecting air voids. As a vehicle tyre passes over the wet surface, the water has paths into which to dissipate without building up the pressures that cause aquaplaning and traffic spray. The spray from vehicles on wet surfaces is uncomfortable and dangerous on busy roads and it is remarkable how the spray vanishes almost completely on encountering surfacings of pervious macadam. They have been found to have another advantage. The voids in the surfacing absorb traffic noise to the extent that, in the Netherlands, these pervious macadams are known as 'whispering asphalts'. Trials are in progress to establish whether these mixtures can be made more durable by using modified bitumens. There is thus a possibility that

TABLE 6.4
COMPOSITION OF DENSE BITUMEN MACADAM BASECOURSES MADE WITH CRUSHED ROCK AGGREGATE (TAKEN FROM BS 4987, 1988)

	Percentage by mass of total aggregate passing test sieve		
Nominal size (mm)	20mm	28mm	40mm
BS test sieve (mm)			
50			100
37.5		100	95 - 100
28	100	90 - 100	70 - 94
20	95 - 100	71 - 95	-
14	65 - 85	58 - 82	56 - 70
6.3	39 - 55	44 - 60	44 - 60
3.35	32 - 46	32 - 46	32 - 46
0.3	7 - 21	7 - 21	7 - 21
0.075	2 - 9	2 - 9	2 - 9
Target bitumen content (% by mass of total mix)*	4.7	4.7	4.5

* Permitted tolerance ±0.6

binders will be developed with the high resistance to weathering that is needed if such pervious macadams are to be used in hot climates.

6.3 Asphalt mixing plants

The original and still the most common type of mixer is the batch mixer in which measured quantities of aggregate and hot bitumen are mixed together in a twin-shafted mixer with contra-rotating paddles. Rudimentary batch mixing plants consist of little more than a loading skip in which the aggregate is apportioned which swings up to discharge into the mixer, the hot bitumen being added via a separate loading trough. Such mixers, using cold and often wet aggregate, can be used with cutback bitumens and emulsions. With penetration grade bitumens it is necessary to dry and heat the aggregate, therefore a rotary drier must be included in the assembly. The different sizes of hot aggregate are separated after heating and stored in separate bins over the weigh hopper which discharges the apportioned hot aggregate into the mixer. A typical layout of a batch mixing plant likely to be installed in a quarry is illustrated in Fig. 6.5. There can be elaborations not shown in this Figure. Whilst once the mixer operator stood on a platform overlooking the mixer and the weigh dials, now he sits in an office operating a computer panel with perhaps 50 or more cards which he can insert to instruct the plant to produce the different mixtures likely to be ordered by his customers. Automation has been carried further, for example, to adjust heat input to the drier according to its load and to adjust the throughput from the cold storage bins so that the hot aggregate bins above the mixer are kept adequately supplied with the different sizes of aggregate. Bitumen supplies to the mixer are metered and, on larger plants, different grades of bitumen kept in separate heaters ready for use when required.

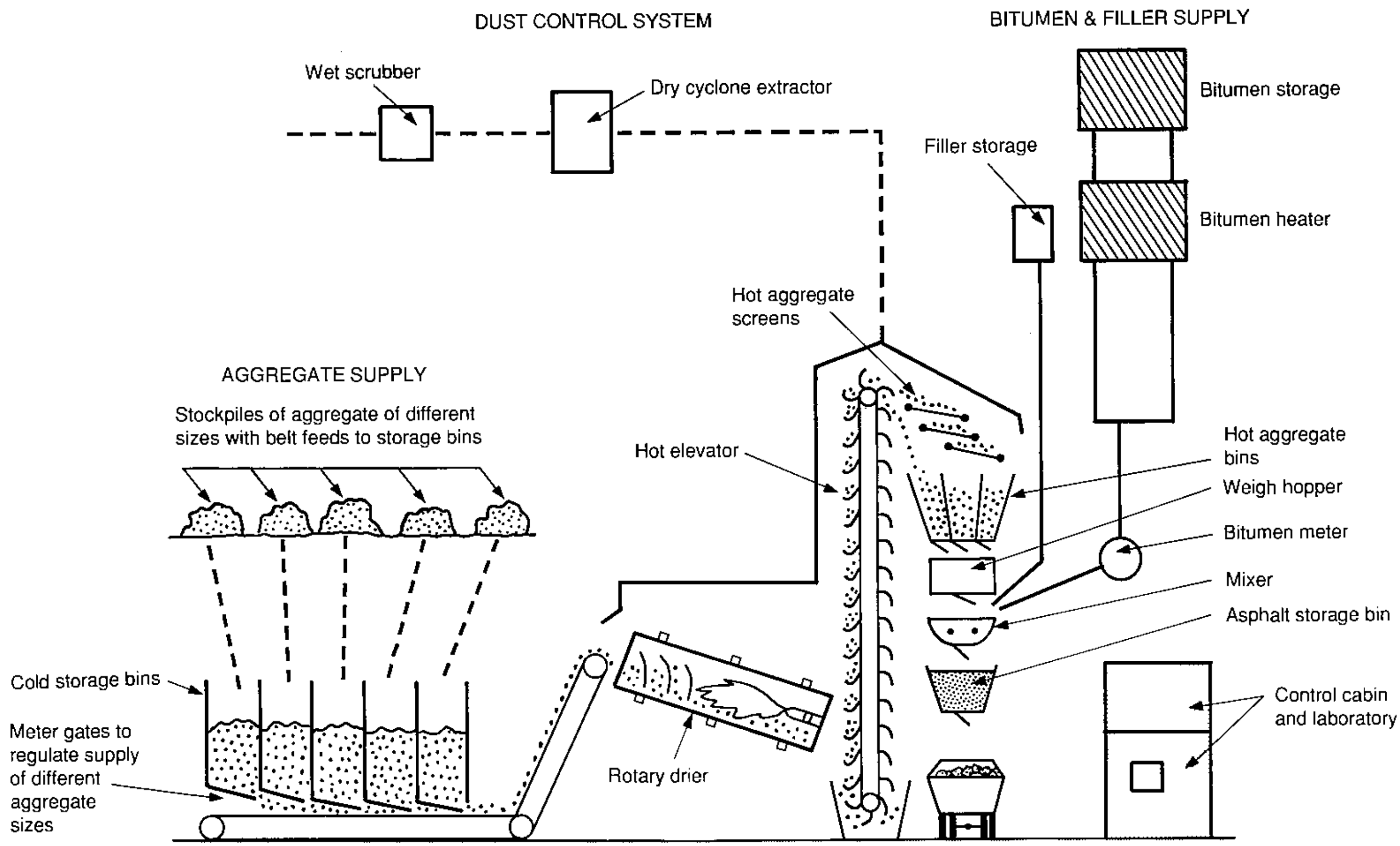


Fig.6.5 *Typical layout of a batch mixing plant.*

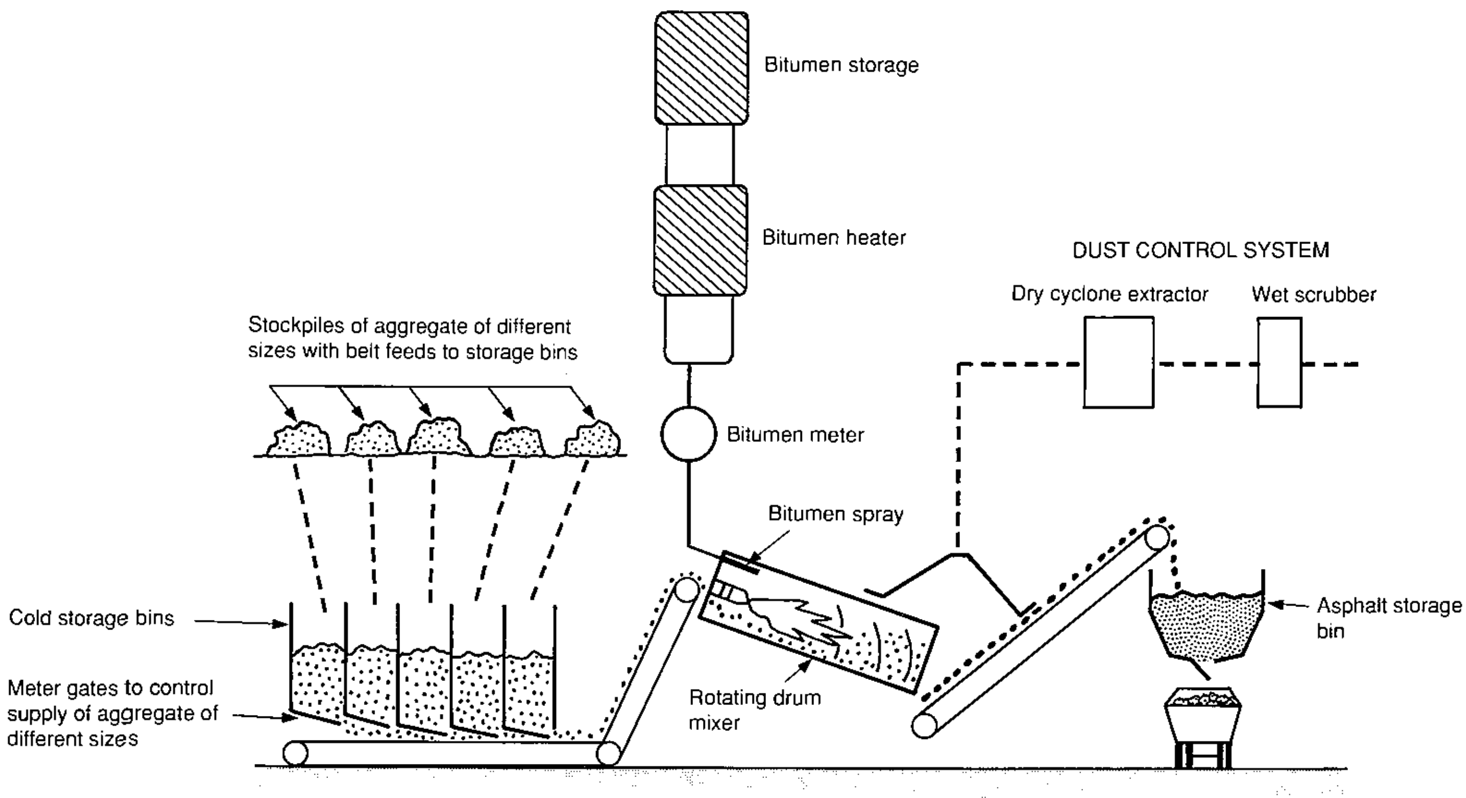


Fig.6.6 *Typical layout of a continuous mixing plant.*

Continuous mixing plants are available which are simpler in form but it is not easy to switch from one mix formulation to another. They have found their main use in surfacing airfields and in other circumstances where large outputs of one asphalt formulation are needed. Over the last 25 years, the drum mixer, a form of continuous mixing plant, has come into increasing prominence (see Fig. 6.6). With these mixers, what was the rotary drier also fulfils the functions of the mixer. The drying flame follows the direction of the aggregate through the rotating cylinder and the hot bitumen is sprayed on to the aggregate as it moves through the cylinder. Such mixing plants are simpler and cheaper to operate than batch mixers. They cannot be used with cutback bitumens or tars since the cutting oils would evaporate and probably catch fire in the mixer. Original misgivings that penetration grade bitumens would also be damaged by the sudden onset of heat in the mixer have been largely set at rest.

With both batch mixers and continuous mixers it is important that the feed gates from the wet storage bins work accurately to deliver the aggregate forward in the correct proportions required in the asphalt mixture. For drum mixers this requirement is paramount since there is no other control over the final grading of the aggregate in the asphalt. With batch mixers there is the additional finer control through the sieves above the hot aggregate storage bins. Some plants, particularly mobile plants used for making rolled asphalt, have additional batch heaters for the apportioned aggregate before it is dropped into the mixer.

The increasing automation of mixing plants has undoubtedly helped in improving both the quality and quantity of output but, as with other forms of machinery when working in remote areas, there comes a nostalgia for the simpler and more robust plant of former years. An asphalt plant with over 30 different electric motors can become an encumbrance without ready supply lines for spare parts.

One of the common needs in developing countries is for patching material to prevent incipient potholes from developing into pavement failures. Quite adequate patching material, namely open-textured bitumen macadams, can be made by skilled operators hand sieving the aggregate and mixing it with cutback bitumen or emulsion using a shovel on a steel plate. Such patching should always be sealed with bitumen and fine aggregate after it has been compacted in place. It is always best to use patching material of similar type and texture to the adjacent surfacing. Where this is asphalt, there may be use for the small, chariot-mounted, spot-mixers available from some plant manufacturers.

6.4 Recycling

On old roads that have had a succession of bituminous treatments which have built up to a substantial thickness, it is common practice to plane the surface to produce a substrate on which new material can be laid. The material planed off usually contains old bitumen or tar and aggregate which can be recycled with new materials to make new surfacings to be relaid on the same road or elsewhere. This process finds particular use on city streets where kerb levels prevent additional layers of premix from being used. Special machinery has been developed, not only to plane the surface without objectionable smoke and smells, but also to mix the planed material with new bitumen and aggregate and relay it on the spot. Alternatively, the planed material may be removed for incorporation in material made in an asphalt plant. Tests on the planed material must be undertaken to determine both the viscosity and the quantity of new bitumen and the amounts of new aggregate needed in the mixtures.

In the Repave process, a single-pass machine planes off about 20 millimetres of the old surface, picks up the planings, remixes them with fresh material in a pugmill and lays the mixture to approximately the same surface levels. Bonding to the hot-planed surface is good and quite satisfactory surface finishes can be obtained (see Mayhew and Edwards (1989)).

In the older Retread process, surfacings are scarified to a depth of about 75 millimetres. The scarified material is then broken down by harrowing and, as the harrowing continues, successive applications of bitumen emulsion are sprayed over the surface. After the bitumen is thoroughly mixed in, the material is recompacted with heavy steel rollers and a surface dressing is then applied. This process can be used to rehabilitate minor roads that have lost their shape after many years of surface treatments and where the relatively light traffic means that strengthening by thickening the construction is not necessary (see Dinnen (1983) and the technical publications of the Road Emulsion Association and the Asphalt Institute).

There may be occasions when recycling can be used in tropical countries. Indeed, one of the earliest examples is the Recondo process developed by N. Taylor (1978) and used in Singapore in the 1930's. On roads with a substantial thickness of grouted macadam, the Retread process offers the prospect of correcting their uneven surfaces. It does not offer any prospects of remedy on roads with thin hituminous surfacings and gravel or stabilised soil bases, and it is not likely to be of value in rehabilitating pavements which lack the intrinsic strength to carry the traffic that is using them.

6.5 Mix-in-place methods

Mix-in-place methods can be used both in making bitumen-stabilised bases with soils of low plasticity and in making road mixtures with crushed rock aggregate. At one stage, such road mixtures were popular in the USA (see the Asphalt Institutes Handbook) but their performance was manifestly inferior to plant-mix with similar ingredients. With the spread of plant-mix facilities over the country, such road mixtures are now rarely employed. Similarly, in France, bitumen emulsions were used with suitably graded natural gravels to make 'grave-bitume' by mix-in-place methods, but this material, used extensively in new road building, is now almost exclusively made in asphalt plants in which the materials can be apportioned with much greater accuracy.

In Scandinavia a particular form of road-mix has been used which takes advantage of the well-graded aggregates that can be found in glacial moraines. With these mixtures, the binder employed was bunker-fuel oil i.e. a crude oil from which the more volatile constituents had been distilled and which is used for firing ships boilers. The hope was that this rather soft binder would harden only slowly so that pavements damaged by frost in winter could be bladed back into shape after the spring thaw. Trials of this process with local lateritic gravel in Uganda were quite unsuccessful. The high surface area and absorptive nature of the clayey gravel meant that very large binder contents (over 10 per cent by weight) were needed to coat the material, making the mixtures far too expensive to be practicable.

Road oiling as a method of laying dust is still carried out occasionally, for instance, where supplies of waste lubricating oils are available, and near ports, where bunker fuel oil may be available. Generally, however, mix-in-place methods for making bituminous surfacings and bases do not seem to be appropriate for use in tropical environments.

6.6 Spreading and compaction

6.6.1 Spreading

Asphalts and macadams can be spread on the road by hand. Indeed, when using premixes to surface awkwardly shaped areas such as in car parks, footways and re-entrant angles on roads, there is often no choice but to spread the materials by hand. Spreading by hand is done with hand rakes and considerable skill is needed to produce a level surface of the right uncompacted depth whilst at the same time preventing the coarse and fine fractions of the mixtures from segregating.

It is impossible for hand spreading to keep up with the deliveries of material from modern asphalt plants. Low temperature premixes can be spread using spreader boxes, even blade-graders, but these are likely to be used only with cruder processes such as mix-in-place methods. Nowadays the mechanical spreader or paver is available all over the world in varying sizes for different purposes. Pavers spread and screed the premix to the width and levels required. The screed can be vibrated so that the asphalt is partly compacted by the paver. The screed operator sets the thickness of material so that when it has been further compacted by rolling, the specified thickness of the layer is obtained. A diagrammatic representation of an asphalt paver is shown in Fig. 6.7.

The further compaction by rolling varies with different materials. With asphaltic concretes and coated macadams it can be quite large, up to 20 per cent of finished thickness. With rolled asphalt surfacings it is usually less.

The screed produces a plane surface. Split screeds make it possible to adjust the transverse camber as required. When laying on a substrate with pronounced hollows and bumps in its surface, these are inevitably echoed to some extent in the surface of the finished compacted asphalt. The initial compaction given by the screed helps to reduce this effect, but when laying surfacings on an uneven substrate e.g. one with inequalities of more than ± 6 millimetres under a 3-metre straight edge, it is wise to fill the hollows beforehand with asphalt. The asphalt for this can be spread by hand. To facilitate feathering-out around the edges of the patch, the asphalt should be made with aggregate of no larger size than 14 millimetres.

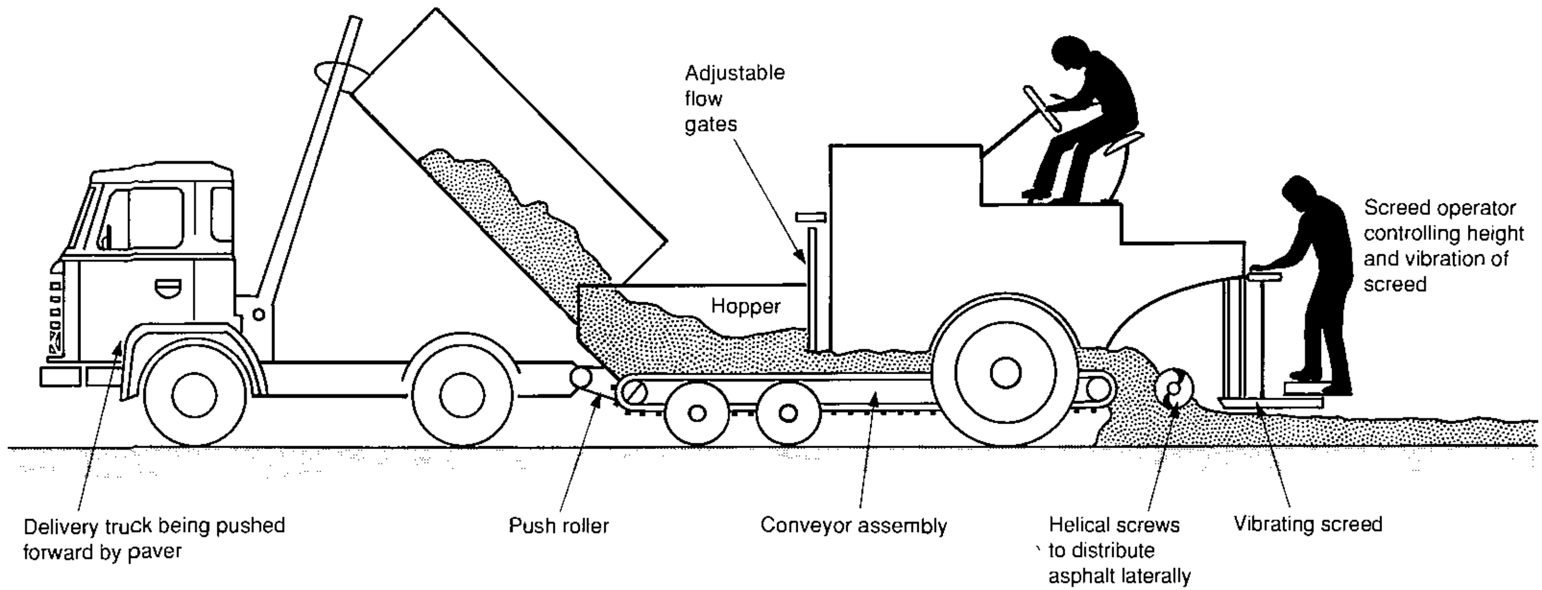


Fig.6.7 *An asphalt paver.*

TABLE 6.5
ASPHALT ROLLING TEMPERATURES

Penetration grade of bitumen in mix	35	50	70	100
Lowest effective rolling temperature °C	90	85	80	75
Upper limit of temperature for rolling °C	135	130	125	120

6.6.2 Compaction

The most usual compaction equipment is a steel-wheeled tandem roller of over 8 tonnes dead weight. Vibratory rollers are coming into greater use and pneumatic tyred rollers of equivalent performance are sometimes used. With these latter it is desirable to finish the rolling with a smooth steel-wheeled roller to remove any inequalities left by the rubber tyres. In cooler climates, thin surfacing mixtures can cool rapidly, particularly in windy weather, and it is vital to start rolling as soon as is practicable after the material is laid. In hot climates, cooling is less rapid and it is important to delay rolling until the asphalt can carry the rollers without rutting. Table 6.5 gives the approximate ranges of mix temperature within which satisfactory compaction can be achieved.

Asphalt pavers do their best work when they are unencumbered by delays and it is always desirable to have at least two full trucks waiting to discharge their load into the hopper of the paver. A radio link between the mixing plant and the works on the road is valuable in preserving this even flow of work and in reducing wastage. Compliance testing involves measuring the densities of the compacted asphalt together with the thickness and surface profile of the finished work. These are considered in Section 6.7.

6.6.3 Joints

Inevitably the hot asphalt has to be laid against hardened compacted material. Care is necessary to make sure that the new material laid against such joints is well compacted and that the joints are well sealed against the ingress of water. On airfields and on new roads, several pavers can work in echelon with compactors following closely behind, thus reducing the need for longitudinal joints. On road resurfacing works, each run of the paver will call for treatment of both longitudinal and transverse joints. Longitudinal edges should be trimmed to a vertical face and should be painted with hot bitumen. Transverse joints are likely to be trafficked at the end of a days work. A temporary ramp of asphalt must be left in place until work is resumed. This ramp must be cut back to a vertical face and painted with bitumen before laying fresh asphalt up against it. Particular care is necessary to obtain good compaction of hot asphalt where this abuts against compacted asphalt that has already cooled. On both transverse and longitudinal joints, a thin layer of hot asphalt should be raked from between the helical screws and the screed of the paver and spread for about half a metre over the adjacent compacted asphalt, as shown in Fig. 6.8. This asphalt is raked back over the uncompacted asphalt and the first passes of the roller are then made over this extra thickness of asphalt along the joint.

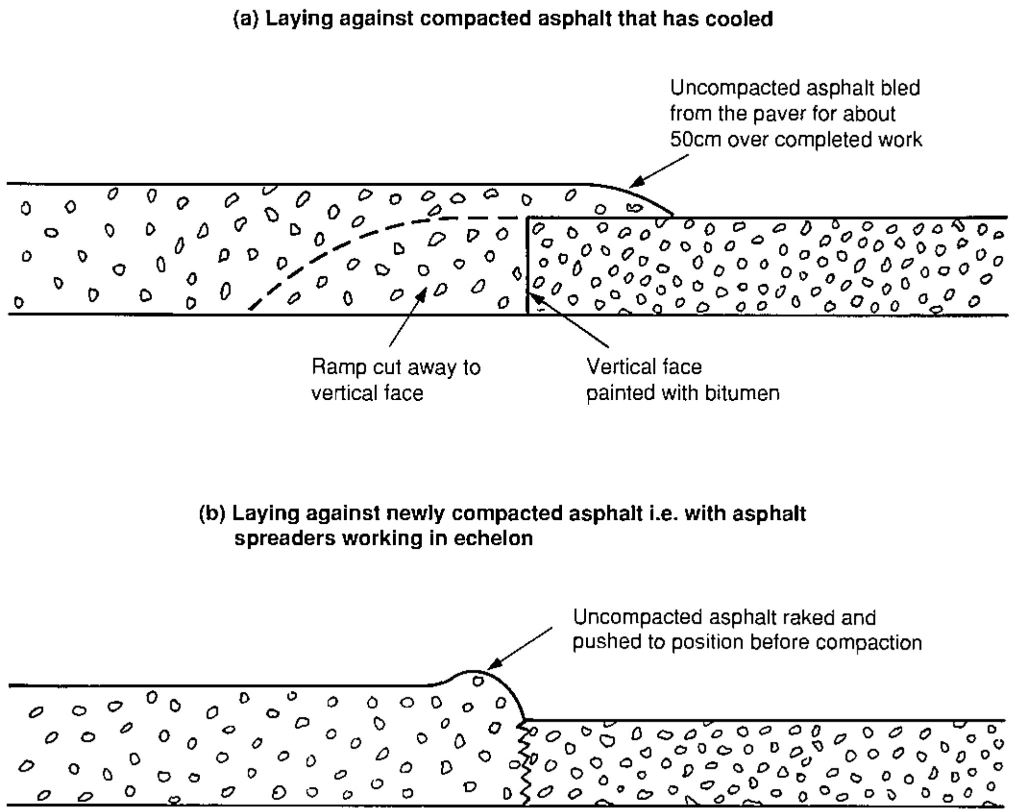


Fig.6.8 Construction of joints in asphalt layers.

6.7 Quality control and compliance

Specifications for bituminous premix normally contain items to define the following properties:

(a) Mix composition
(b) Minimum density of compacted material
(c) Nominal thickness of layer
(d) Evenness of finished surface profile

These four are the essential items. There may be others, for example, minimum texture depth for busy roads carrying high speed traffic where a high resistance to skidding is required, minimum delivery temperature to insure that the material can be adequately compacted, restrictions on weather conditions when work can proceed, and, for work done on roads that are kept open to traffic, a definition of the safety precautions that must be taken as a defence against accidents.

Most of these items are covered in the National Standards for the different premixes and, in ordering a material from a contractor, it is sufficient to designate the particular mixture required. It is still necessary for the customer to seek assurance that he is obtaining what he has ordered. This is not an easy business. It is quite impossible to make sure that every small component of the work conforms to a specification. Most of the tests require considerable care to make sure that accurate results are obtained but speed in testing is necessary to make sure that faults are corrected before much faulty material is laid.

The increasing automation of asphalt mixers has helped to remove some of the errors that occurred when the apportioning process was entirely under manual control. But errors still occur due to the malfunction of particular mechanical or electrical components, and changes in the grading of the aggregate delivered to the mixer occur as the quarrying machinery is adjusted to meet the varying demand for stone of particular sizes. With the larger outputs from modern plants, it is even more vital to detect quickly faults that have developed.

A distinction must be made between quality control and compliance testing. Quality control is an ongoing process in which the manufacturer seeks to correct, or even anticipate, faults in production. He will undertake regular sampling and testing of the grading of the aggregate as it goes through the plant. He will check temperature control appliances. He will make regular checks on the accuracy of the apportioning gear, the sieves and the metering devices for controlling the proportions of aggregate and binder, and he will regularly sample the output to make sure that the specified mix proportions are being maintained. On the road, he will supervise the operation of the laying and compaction machinery to make sure that it is working satisfactorily. His test methods may involve short cuts but, though they may lack intrinsic accuracy, they are generally good enough to indicate rapidly when things are going wrong.

Compliance testing has a different purpose. It has a contractual significance to determine whether the supplier is meeting the requirements of the customers' specifications. For this purpose it is of primary importance that the tests are accurate. Furthermore, enough testing must be done to make sure that the results give a reasonable indication of how well the laid material conforms to specifications. What does reasonable mean? It ought to mean that the risks are equally distributed between the supplier and the customer i.e. that the risks to the consumer that he will accept undetected defective material are equal to those of the supplier that large areas of work will be condemned on the evidence of tests on only small quantities of material.

This is beginning to sound like an abstruse and rather academic point of engineering philosophy and, indeed, it may go beyond what is currently important in some developing countries. But disputes over the interpretation of contract documents occur in the developing world as well as in industrialised countries and the recourse to law or arbitration is usually an expensive and tedious business.

In some countries, lawyers are seeking to complicate the situation still further by insisting that suppliers have a responsibility beyond the requirements of the specification to provide material which is 'fit for use'. What a happy and lucrative time they can have over that. Far better if suppliers and customers are both aware of the uses and limitations of compliance testing and can work together to produce constructive solutions to problems when they arise.

There are several ways in which there are prospects of easing these problems. One involves pushing the 'fitness for use' concept to its limit i.e. by specifying only the qualities of the end product required. Some of these qualities cannot readily be evaluated, resistance to ageing, for instance, or the contribution that a surfacing makes to structural strength. In effect, the 'fitness for use' concept involves the supplier in guaranteeing the performance of the surfacing for a number of years. This is done in Switzerland, for example, and has been done elsewhere in the past. It usually involves a free maintenance period of, say, five years, with a maintenance contract for a further five years. Ten years is a long period in the life of a latter day contractor and the legal eagles are now well aware that the behaviour of a surfacing can be critically affected by the strength of the pavement below.

Another way lies in the possible development of a simple and reasonably rapid testing regime which is more reproducible than the Marshall Test procedure and which gives a more reliable indication of the basic properties required in bituminous premixes in different situations. Anyone who successfully launches such a test is sure of an honoured place in the annals of highway engineering.

A further line of development lies in providing direct incentives for suppliers to operate effective systems of quality control. It is common practice in many countries for highway authorities to operate a system in which contractors have to prove their competence on small works and are permitted to compete for larger works only if their compliance with specifications reaches a satisfactory level. In some countries this concept is being formalised. Contractors have to demonstrate that they are following defined procedures in looking after their processing machinery, in checking the accuracy of their manufacturing processes and in testing their products. Inspectors call on them at intervals to make sure that the procedures are being followed and government contracts are awarded only to firms which accept and work to the prescribed condition. Such a scheme, described by the Australian Road Research Board (1986), is in operation in that country. In the United Kingdom a scheme is currently being introduced to cover the operations of materials testing laboratories run by contractors, consultants and highway authorities.

Another incentive involves defining payment penalties in the contract which will be exacted for work which falls only slightly out of specification but which has a diminished value in terms of the service it will give. This is logical because, with most road making processes, there is no sudden change in the service given by a material from good to bad as its composition moves across the limits of a specification.

In effect, such penalty clauses are a method of formalising the debates which frequently develop between supplier and customer, both seeking to avoid the penalty in cost and delay of removing large areas of material which are manifestly, but only slightly, out of specification. The work on quality control in Australia has produced a statistical basis for introducing a graduated payment scheme covering the state of compaction of asphaltic concrete. This is described by Rebecchi and York (1981) and an example of its use by the Country Roads Board of Victoria is shown in Table 6.6. In this example a 'lot' is usually a day's work from which six to ten samples are taken for density measurements. The factor k_0 is included to take account of the inherent variability in the test method. Where direct measurements of density are done on cores, a value of 0.92 is assigned to k_0, as in the Table. When nuclear methods of measuring density are used, which are somewhat less accurate, a lower value of 0.88 is used.

TABLE 6.6
SPECIFICATION FOR RELATIVE COMPACTION LEVELS FOR A LOT OF ASPHALTIC CONCRETE (AFTER DICKINSON (1985)) (MEDIUM AND LARGE-SCALE WORKS)

Nominal thickness of layer (mm)	*Relative density (%) minimum acceptance level without penalty*	*Payment penalties if below minimum acceptance level*
<30	$\overline{X} = 93$	—
30 to 50	$\overline{X} - k_o.s = 93$	(a) If above 90%, graduated penalty
>50	$\overline{X} - k_o.s = 95$	(b) If below 90%, option of rejection

$\overline{X}$ is the mean value of six to ten test results
s is the standard deviation of the test results (estimate of population standard deviation).

6.8 Analysis methods for premixes

Whatever method of mix design is used for its preparation, the premix is defined in terms of the grading of the aggregate and the bitumen content of the mix. This latter is usually defined as the percentage by weight of the total mix. These are the quantities used in operating the apportioning devices on the asphalt mixer and it is these quantities that are used in determining how well the material conforms to specification. There has been constant pressure to reduce the time taken to complete such analyses so that the tests can be useful in production control as well as for determining compliance with specification. The tests are described in the following British Standard:

BS 598 (1989). Sampling and examination of bituminous mixtures for roads and other paved areas. Part 102. Analytical test methods.

These analytical tests are listed in Table 6.7 which shows how the time taken to complete the analysis has been successively reduced. With the sieving extractor illustrated in Fig. 6.9, it seems likely that the ultimate has been reached in the speed with which such analyses can be done. All the test methods have been carefully evaluated to determine their repeatability and reproducibility. Both are quite good and these are currently the only methods which can be confidently used to establish whether the composition of premixes conforms to specifications.

6.8.1 Sampling

Quite large errors can be introduced by careless sampling and it is important that the sampling and quartering procedures described in the Standards be carefully followed. Segregation of coarse and fine aggregate can be a difficulty, both in sampling and in laying asphalts on the road, but some minor segregation can be tolerated. When it occurs, the bitumen will tend to follow the fine aggregate because of its larger surface area per unit weight. For example, for the aggregate divided on the 2.36 millimetre sieve, the fine fraction will carry about 2.5 times as much bitumen

TABLE 6.7
METHODS OF ANALYSIS OF BITUMINOUS PREMIXES

Method	*Solvent*	*Bitumen content determined*	*Filler content determined*	*Time to complete test*
Funnel	Trichlorethylene or methylene chloride	by difference	directly	2 - 4 days
Hot extractor	Trichlorethylene	by difference	directly	2 - 4 hours
Extraction bottle	Trichlorethylene	by difference	directly	1 - 2 hours
Extraction bottle	Methylene chloride	directly	by difference or directly	1 - 2 hours
Sieving extractor	Methylene chloride	by difference or directly	by difference or directly	30-50 mins

The solvents used in these methods are somewhat toxic and the tests should always be carried out in well ventilated fume cupboards.

as the coarse fraction, weight-for-weight. This should be taken into account in interpreting the results of the analysis i.e. the target bitumen content should be adjusted according to the fines content found on analysis. The correction required to the target bitumen content can be derived from the expression,

$$\Delta b = \frac{3(a - n)b}{3n + 200}$$

where Δb is the change in bitumen content (per cent by weight of total mix)
b is the target bitumen content (per cent by weight of total mix)
a is the percentage of fine aggregate found by analysis
n is the target percentage of fine aggregate by weight

An example will clarify the significance of this correction. A typical rolled asphalt would have a target of 55 per cent by weight of fine aggregate associated with a target bitumen content of 7.0 per cent. Should analysis reveal 60 per cent of fine aggregate (within normal mix tolerances), the target bitumen content should be increased by,

$$\Delta b = \frac{3(60-55) \times 7.0}{3 \times 55 + 200} = 0.29$$

indicating that the target bitumen content is 7.29 per cent for this sample.

Samples should be taken for analysis at least once a day on any particular contract, more frequently on asphalt plants of high capacity. Even so, tests on a sample weighing less than one kilogram are taken as representing the quality of several hundred tonnes of mixed material. Such testing rates may not be abnormal in engineering processes where both the materials and the

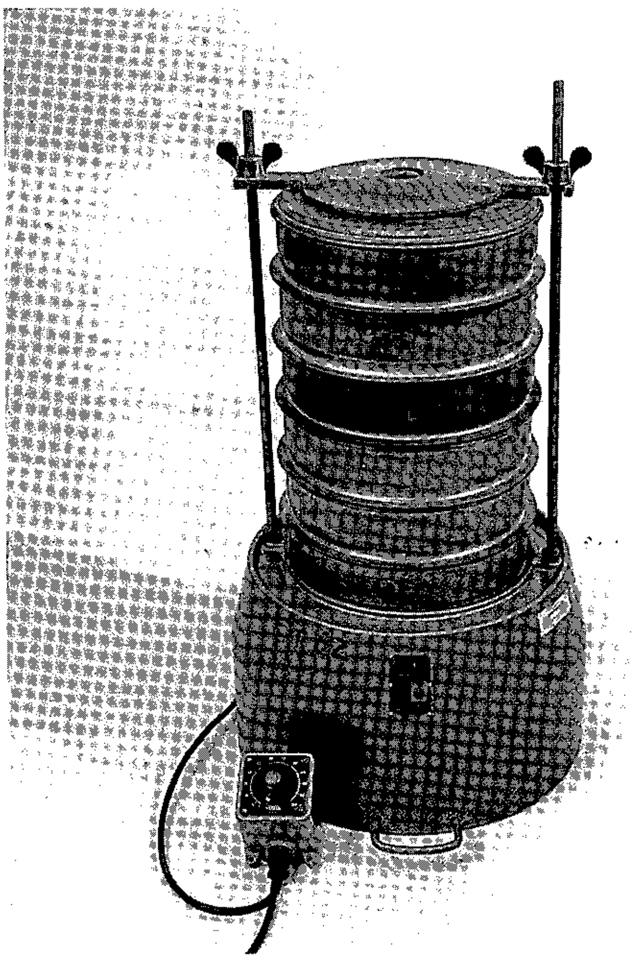


Fig.6.9 The sieving extractor.

methods of production are highly refined but it does seem rather inadequate with a material of which the major ingredient is crushed rock processed with machinery that has to withstand the highly abrasive action of the rock as it is crushed. The search is still on for methods of testing which produce an instantaneous and continuous record of the composition of bituminous road mixtures. Bitumen contents can be indicated by nuclear methods (see ASTM D4125(83). Standard test method for asphalt content of bituminous mixtures by the nuclear method). Calibration is necessary with different aggregates and results can be prejudiced by any water present in the mixture. The apparatus is similar to that illustrated in Fig. 2.9 for measuring the dry density and moisture content of soils. It can be useful in quality control and can be acceptable for compliance testing if well calibrated and provided that regular checks of composition are made by analysis.

6.8.2 Compacted density

The simplest method of specifying the compaction of premix is by defining the types of roller that can be used, the number of passes, and the range of rolling temperatures. BS 594, Part 2, defines a procedure by which the performance of different types of roller can be evaluated in comparison with the traditional 8-10 tonne tandem steel-wheeled roller. Consistently higher densities have been obtained with vibratory rollers. It is particularly important to make sure that the required number of roller passes is made on the material in the wheel tracks.

As the importance of thorough and uniform compaction has been realised, end product specifications have come into increasing use, particularly with material laid on more heavily trafficked roads. There are two forms of end product specification:

(a) Maximum and minimum air voids.
(b) Density in comparison with laboratory compacted specimens.

The measurement of air voids requires accurate determinations of the composition of samples and of the specific gravities of all the ingredients. It is a useful means of monitoring the compaction of premixes under traffic but the work involved does not commend this method for either quality control or compliance testing.

Measurements of compacted density in comparison with densities obtained in the course of Marshall design testing in the laboratory are used all over the world, with minimum densities specified as a percentage, usually 93 to 98 per cent of the maximum density obtained in the testing. This method works quite well with asphaltic concrete surfacing mixtures but, as Powell *et al.* (1980) established, the compacted densities obtained in the Marshall compaction procedure depend on different composition variables to those influencing compaction of mixtures on the road.

The Percentage Refusal Density (PRD) Test appears to overcome these difficulties with the additional advantage that it can be used on dense bituminous basecourse and base mixtures as well as on dense asphalt surfacings. In this testing procedure, the measured densities of cores of asphalt taken from the road are compared with the maximum density that can be achieved in the laboratory on the material in these cores under standard heavy loading. The test is described in BS 598, Part 104 (1989) and experience in using the test has been described by Powell and Leech (1982).

The test gives consistent results and is an appropriate end product method of specifying the state of compaction of all dense bituminous road mixtures. Guidance on selecting appropriate levels of PRD for different mixtures is given in BS 598.

A disadvantage is that the test takes up to three days to complete, occupyiug a laboratory technician for about seven hours over that period to test six samples. This limits the value of the test for day-to-day monitoring of production, particularly when there is a risk that premix bases and basecourses may be covered by succeeding courses of material. This difficulty can be overcome by the use of nuclear densimeters for day-to-day monitoring, with the PRD test being used to arbiter compliance with the specification.

6.8.3 Thickness of compacted layer

The cores removed for density measurements may also be used to check that the thickness of layer corresponds with that specified. Thicknesses are likely to vary, particularly with material laid on an uneven substrate. It is normal to require that the specified thickness be obtained or exceeded in at least 95 per cent of the measurements. Payment is usually made at a price per square metre and contractors may have a merited claim if they find that an uneven substrate means that they have to supply more material than expected to produce the required minimum thickness. In such cases it is usual to check the merit of a claim by comparing the tonnage of material laid against that expected had the material been of uniform specified thickness.

6.8.4 Surface profile

Mobile profilometers are available for measuring surface profiles. One of these, the rolling straight edge, can be used in measuring the finished profile of roads but it is normal to use an ordinary hand-held straight edge. Typical permitted tolerances are given in Table 6.8.

TABLE 6.8
TYPICAL TOLERANCES ON FINISHED PROFILE AT DIFFERENT LEVELS IN PAVEMENT CONSTRUCTION

Pavement course as constructed	*Maximum distance between surface and underside of 3m straightedge*
Pavement surface	3mm
Bituminous basecourses and bases to carry premixed surfacings of 50mm or less in nominal thickness	6mm
Sub-bases and bases to carry two-course bituminous surfacings of total thickness greater than 50mm	10mm

Other mobile profilometers find more use in surveys to check the condition of existing roads. Here, the objective is to determine the riding quality of the surface. The Bump-Integrator, illustrated in Fig. 6.10, is normally used for this purpose. This machine employs a single-wheeled trailer towed behind a suitable vehicle travelling at fixed speeds. The wheel is instrumented to measure its vertical displacements in one direction. These are summed over a fixed distance of travel, giving an index of surface irregularity which is recorded as total displacement per kilometre at the designated speed of travel. This measurement of riding quality is sometimes known as the roughness of the surface. In this use of the term roughness there is risk of confusion with the rugosity of surface required to produce a high resistance to skidding on high speed roads. It is therefore always desirable to make sure of the sense in which the term roughness is employed.

Fig.6.10 The towed fifth-wheel bump integrator.

6.8.5 Resistance to skidding

An end product specification for resistance to skidding is not possible since road surfacings develop their critical resistance to skidding only after they have been trafficked. Furthermore, the resistance to skidding can vary with varying weather conditions in the immediate past. Two parameters are used in specifying the anti-skid characteristics. One is the propensity of the exposed aggregate to polish under the action of traffic. For this purpose the Polished Stone Value Test is used on the aggregate (see Chapter 4). The other is the depth of the texture in the surface as laid, a high texture depth being necessary on busy roads carrying fast traffic to allow water to escape from between vehicle tyres and the wet road surface.

Texture depth is conveniently measured by the Sand Patch Test. In this test, a measured quantity of sand of standard particle size is piled on the road surface and is gently manipulated into a

TABLE 6.9
TYPICAL TEXTURE DEPTHS FOR DIFFERENT ASPHALT SURFACINGS

Material	*Normal range of texture depth (mm)*
Asphaltic concrete	0.4 - 0.6
Dense bitumen macadam	0.6 - 1.2
Rolled asphalt with precoated chippings	0.5 - 2.5*
Pervious macadam	1.5 - 3.5
Surface dressing	2.0 - 3.5

* With heavily chipped rolled asphalt and mastic asphalt, the texture depth normally exceeds 1.2 mm.

circular shape using a standard rubber-faced pestle. The circle is increased in size until the high points of the exposed aggregate are just exposed. The mean diameter of the circle is then determined and, from this, knowing the volume of sand used, an average texture depth can be calculated. Each form of asphalt surfacing has a typical range of texture depths deriving mainly from the grading characteristics of the aggregate used. Typical figures are given in Table 6.9.

A contactless sensor has been developed for measuring texture depth by the Transport and Road Research Laboratory. This sensor uses a pulsing laser and the principle on which it operates is shown in Fig 6.11 (Roe *et al.* (1988), and Hosking *et al.* (1987)). The sensor is available in two forms. One is the high speed texture meter (HSTM) which can operate whilst travelling at normal traffic speeds and which is used in surveys of the condition of existing roads. The other is the

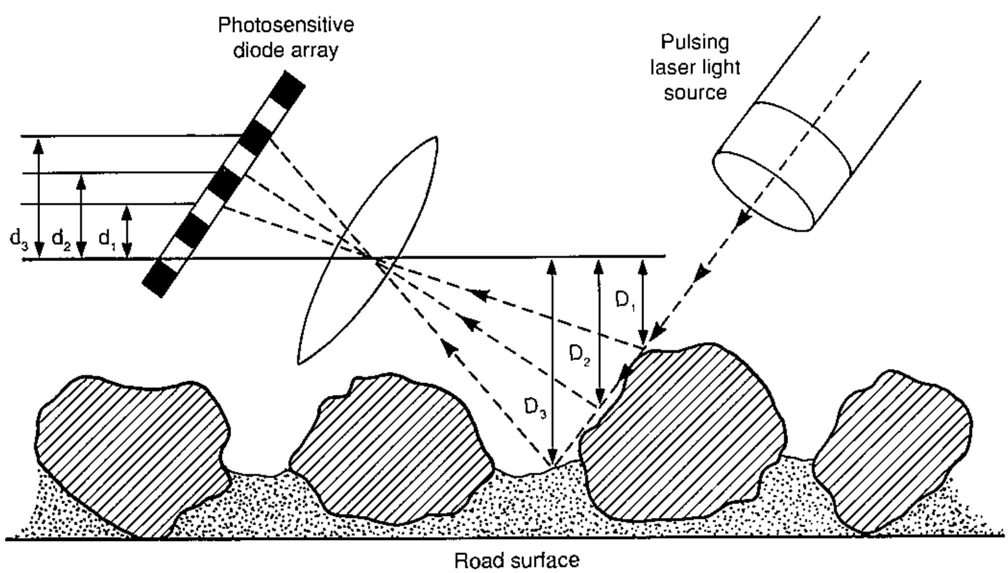


Fig.6.11 The contactless laser sensor for texture and profile measurements.

mini-texture meter (MTM), a manually propelled meter mounted between two wheels. This provides more consistent and rapid measurements than the sand patch method.

Correlation with sand patch measurements varies according to the geometric characteristics of the type of surfacing. So far, a correlation has been established with the heavily chipped rolled asphalt surfacings used on heavily trafficked roads in the United Kingdom and is as follows:

HSTM measured texture = 0.12 + 0.59 x (Sand patch texture depth)
MTM measured texture = 0.41 + 0.41 x (Sand patch texture depth)

With these heavily chipped rolled asphalts, a minimum sand patch texture depth of 1.5 millimetres is currently specified corresponding to a minimum of 1.03 millimetres MTM texture depth. Such a high texture depth is required only on high speed roads. At other sites where skidding accidents are likely to occur at speeds of 50 km/hour or less e.g. at junctions and roundabouts in urban areas, the coarse surface texture is not needed and attention must be focused on obtaining a surface with a rough micro-texture.

Direct measurements of resistance to skidding are made during surveys to determine the current condition of existing roads. Many forms of skid resistance tester have been produced including vehicle decelerometers, hand-operated pendulum devices and braking force meters attached to towed fifth wheels. It is now recognised that the most reliable method is by measuring the sideways force i.e. the lateral force generated by a freely rotating towed wheel inclined at an angle to the direction of travel. The ratio between this force and the load applied to the wheel is the Sideway Force Coefficient (SFC). It is important that the SFC be measured at the typical traffic speed on the road being tested. Fig. 6.12 shows the relationship between SFC and speed for surfacings with coarse and fine surface textures and illustrates the different effects of speed on the skidding resistance of surfacings with different macro-textures. SCRIM, the SFC machine developed by the Transport and Road Research Laboratory, is shown in Fig. 6.13. This machine carries its own supply of water for wetting the road before testing and contains computerised recording equipment which can be made compatible with the data storage systems used in the surveys of current road conditions now undertaken in many countries to help in determining road maintenance needs.

	Texture	
	Macro	Micro
A	Rough	Harsh
B	Rough	Polished
C	Smooth	Harsh
D	Smooth	Polished

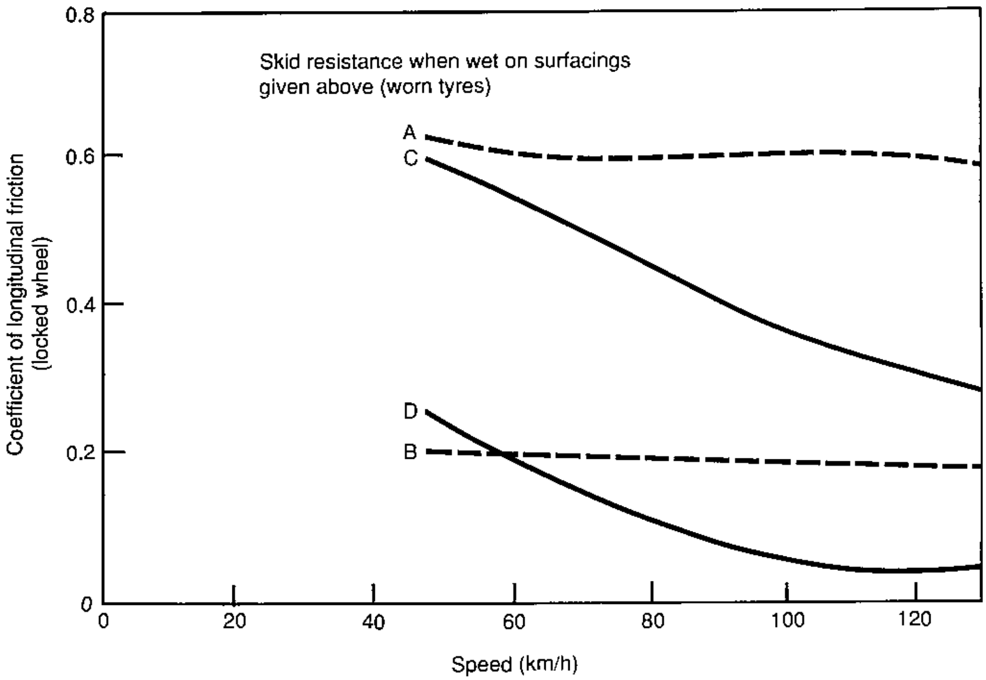


Fig.6.12 *Relationships between coefficient of friction and speed for different surfaces.*

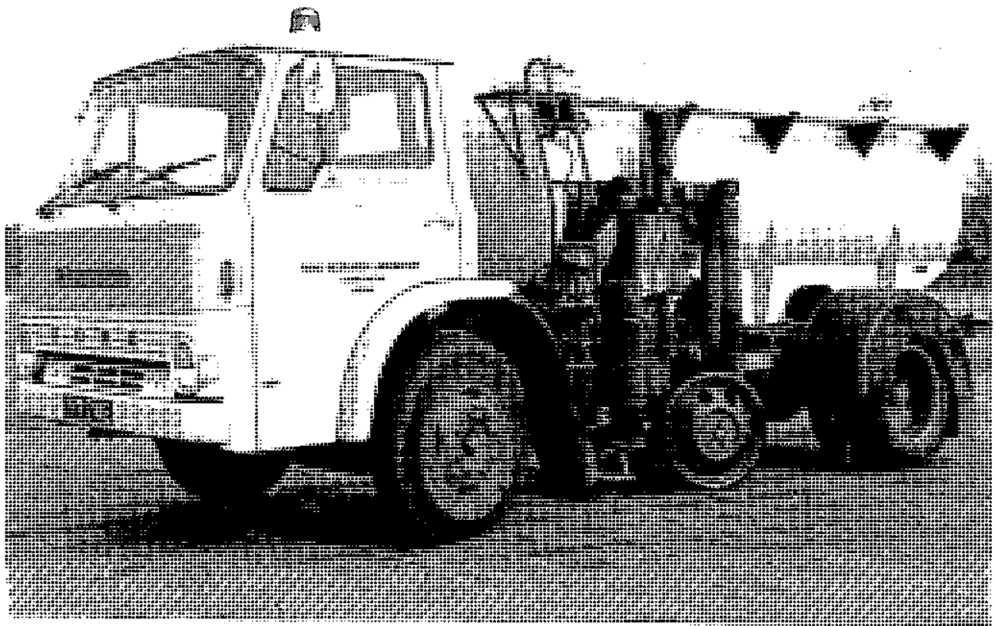


Fig.6.13 SCRIM, the machine for measuring Sideways Force Coefficient (SFC).

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7 Surface dressing

7.1 Introduction

Surface dressing goes by several names in different parts of the world, surface treatment in the Americas, seal and chip, and even tar-spraying. Whatever it is called it is one of the most important weapons in the armoury of the road engineer. On more lightly trafficked roads it provides the most appropriate initial bituminous surfacing. On all roads it is often the most effective method of preserving a deteriorating pavement. When well done, it is quite durable. In Australia, surface dressings on rural roads last for ten years or more before further treatments are required. In Europe it has featured prominently in providing the all-weather surfacings that now cover all the roads in many countries. In the United Kingdom it is now being used to improve the surface texture of concrete motorways that have worn smooth under heavy traffic. And it is by far the cheapest of all forms of bituminous surfacing.

In concept it is a simple process. It comprises the spraying of a film of bitumen or tar on the road surface followed by the application of a layer of stone chippings. The bituminous film seals and binds the road surface and the chippings provide a durable and non-skid running surface for traffic. Surface dressings do not restore the riding quality of an uneven road surface nor do they make a direct contribution to the structural strength of a pavement. In new construction their purpose is to provide a durable and waterproof running surface. In maintenance they have several functions. One is to restore the non-skid properties of a surfacing that has become slippery, but the most important lies in their use to preserve the structural integrity of flexible pavements against the destructive action of traffic and weather.

In hot climates it is usually the upper layers of pavements that deteriorate most rapidly. The bitumen binder in the surfacing weathers more rapidly in the prevailing high temperatures than in temperate zones. This weathering is particularly rapid in bituminous premixes in which the bitumen is present as only a thin film on the aggregate. In an asphaltic concrete made with 80-100 pen. grade bitumen, the bitumen near the exposed surface can weather to less than 20 pen. within two or three years (Fig. 5.14). Such weathering frequently becomes evident as a network of fine cracks in the surface of the asphalt. The asphalt loses its ability to accommodate the strains imposed by the transient deflections of the pavement under heavy traffic loads and, in climates with large diurnal changes in temperature, cracks develop in the surfacings to relieve thermal stresses. It is quite common for such signs of pavement distress to appear within the first five years of the life of a new pavement.

In arid climates these cracks may spoil the appearance of the road surface but they are of no immediate engineering significance. But in climates where the annual cycle includes periods of heavy rain, failure to seal these cracks can have expensive consequences. Water, entering through the cracks, accumulates in the upper layers of the pavement. Before it has time to drain away, powerful hydrodynamic stresses are generated under traffic loads, potholes develop and spread rapidly. Much of the value of the investment in the road can be lost in early disintegration of the pavement.

Herein lies the particular value of surface dressing. In surface dressings the bitumen is present in much thicker films than those in premixed bituminous surfacings. Several advantages follow from this. Surface dressings are immediately effective in sealing any cracks against the entry of water. Because the bitumen film is relatively thick, the surfacing is better able to accommodate transient deflections under traffic without cracking. There is also evidence that the thick binder film in a surface dressing protects the bitumen in an underlying premix from the rapid weathering that can occur in exposed premixes (Smith *et al.* (1990)).

In many developing countries, the large investment in new road building over the last four decades has not been backed up with a comparable effort in maintenance to prevent the road pavements from deteriorating. Surface dressing is the most important component in such preventative maintenance. It seems likely that, because of the more rapid weathering of bituminous surfacings in hot climates, surface dressing has an even more important role than it has had in building up the networks of bituminous surfaced roads in countries with more temperate climates. There are reasons why preventative maintenance has been inadequate in many countries. One is that transport economists find difficulty in assessing its value in the systems they have developed for determining priorities in road maintenance and rehabilitation. These difficulties and possible remedies are considered at the end of this chapter.

There are many excellent publications describing in detail the techniques of surface dressing. A selection of these is included in the references to this chapter. What follows is a review of the essential features involved in the design and execution of the different forms of surface dressing.

7.2 Design of surface dressings

In new construction and road rehabilitation, the bituminous surfacing on roads carrying up to 1000 or so vehicles a day can be by surface dressing. This involves a prime coat on the finished base followed, usually, by a double surface dressing i.e. a second application of bitumen and chippings on top of the first. This provides a more robust surfacing than a single surface dressing with the advantage that any deficiencies in the quality of one dressing will be at least partially concealed by the other.

In treating existing roads, a single surface dressing will normally be adequate but there may be occasions on more important roads where a more elaborate treatment is desirable.

Detailed recommendations for the surface dressing of roads in tropical and sub-tropical countries are contained in Overseas Road Note No. 3 issued by the Transport and Road Research Laboratory. There is also much to learn from the specifications issued by Australian State Highway Departments. The practice in the United States of America is described in publications of the Asphalt Institute.

7.2.1 Choice of binder

Penetration grade bitumens, cutbacks and bitumen emulsions can all be used for surface dressings. Penetration grade bitumens and cutbacks must be applied hot so that, after they are sprayed on the road, they are sufficiently fluid to develop good adhesion with the chippings. Fig 7.1, adapted from Overseas Road Note No. 3, shows how ambient road temperatures affect the choice of bitumens for surface dressing. The softer penetration grade bitumens can be used in very hot climates but cutback bitumens will be preferred for most circumstances. Either medium or rapid curing cutbacks can be used in surface dressings on new construction. For surface dressing existing roads, where it is important to be able to open the roads to traffic as soon as possible, rapid curing cutbacks should be used.

Bitumen emulsions are usually applied cold but, on roads carrying very heavy traffic, there can be value in using emulsions of low water content which are made more fluid by gentle heating before they are sprayed. The choice between cationic and anionic emulsions will depend on the mineralogy of the chippings i.e. cationic for use with igneous rocks and anionic with calcareous rocks. Quick breaking (labile) emulsions should be used.

The use of bitumens containing natural rubber has proved advantageous in surface dressing. It inhibits the flushing-up of the binder around the chippings, which can occur during periods of particularly hot weather, and it also helps to hold the chippings in place. As reported by Hoban (1990), the use of bitumens modified with other polymers is being explored for surface dressing extremely heavily trafficked motorways and other roads.

In new construction, a prime coat of bitumen is applied to the surfaces of bases made from gravel, crushed stone or stabilised soil before any bituminous surface is laid on them. The purposes of this prime coat are to bind the surface of the base and to secure good adhesion between base and surfacing. Fluid cutback bitumens with viscosities of the order of 100 centipoises at ambient temperature are used for this purpose so that they can be applied without heating. It is an advantage for the base to be slightly damp when the prime coat is applied. The fluid bitumen then penetrates into the base for five millimetres or so. The ASTM grades of cutback, MC 30 and MC 70, are usually appropriate for this purpose. Hitch and Stewart (1987) describe how such cutbacks can be prepared on the spot. Although bitumen emulsions can be applied at ambient temperatures, they are not suitable for priming because they coagulate to produce a thin film of bitumen on the surface without the necessary penetration into the base.

7.2.2 Choice of aggregates

Hard, tough chippings are particularly desirable for surface dressing. A maximum Aggregate Crushing Value of 25 is desirable, with harder chippings preferred on more heavily trafficked roads. Where only weaker aggregates are available, quite good results can be obtained provided the roller used is rubber-shod since it is under the loads of steel rollers that the chippings are most prone to fracture. In wet climates it may be desirable to look for chippings with the highest possible Polished Stone Value for use on roads carrying heavy traffic.

Chippings of single nominal size should be used. The nominal sizes of chippings used in surface dressing are 6, 10, 14 and 20 millimetres and it is important to make sure that they have no more

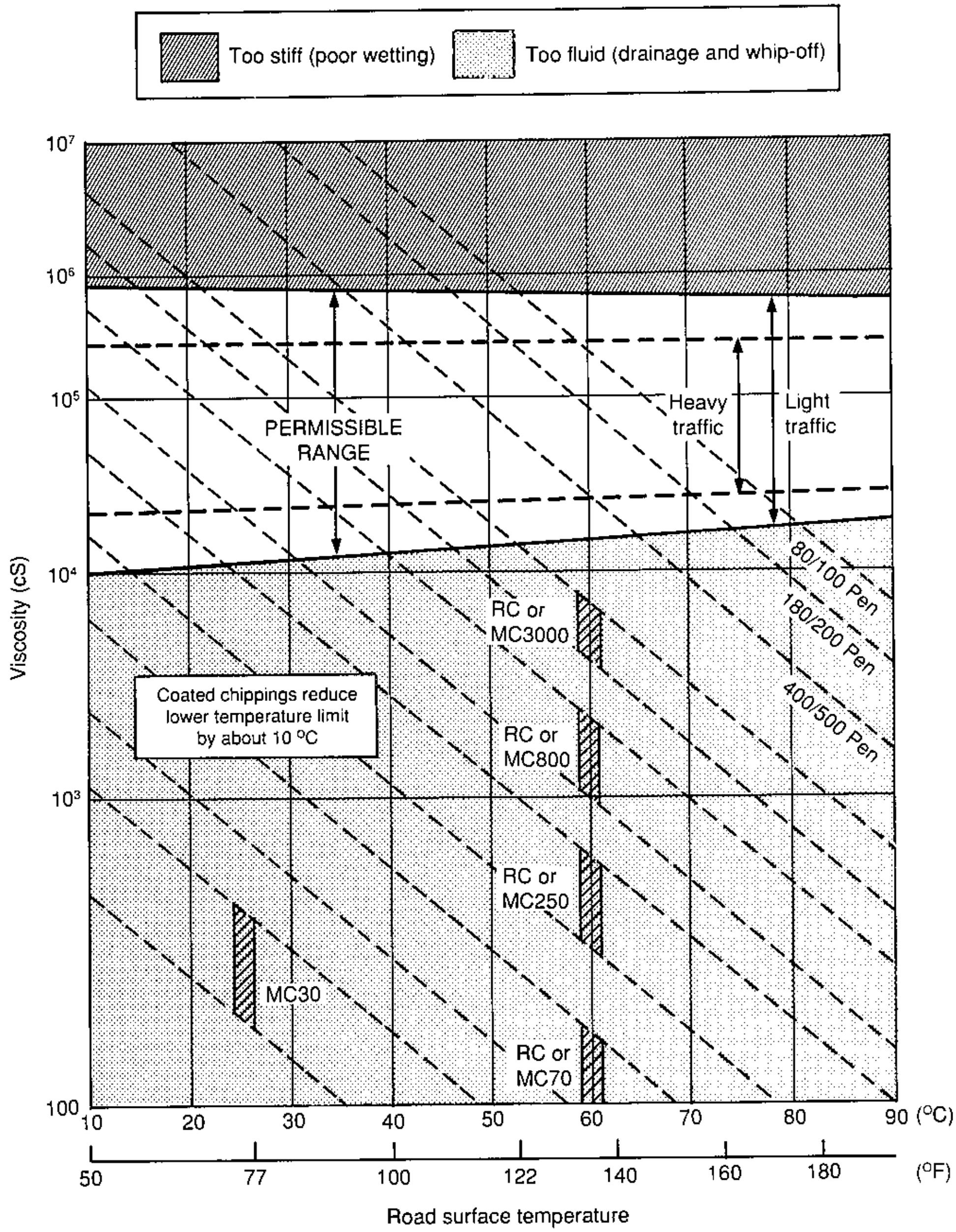


Fig.7.1 *Temperature-viscosity chart to show how road surface temperature affects choice of binder for surface dressing.*

oversize or undersize than is permitted in the standard specifications for the grading of single-sized crushed aggregates. Flaky and elongated chippings are to be avoided since these move about under the roller and under subsequent traffic, and do not remain in the closely packed mosaic of chippings which is the objective in good surface dressing. Dusty chippings are a hazard which seems to be particularly common with crushed rock in hot climates. If dust settles before the chippings on the newly sprayed binder, the chippings will fail to adhere. Fortunately, with mechanical chip spreaders which have a transverse delivery gate, the chippings fall more rapidly than the surrounding dust and are thus able to make good contact with the binder film. Dusty chippings that are also slightly damp present more of a problem. With excessively dusty chippings it may be desirable to wash them in the stockpiles or to pre-coat them in a small mobile mixer with either a light oil or with a thin coating of hard bitumen before they are used. Such precoated chippings should not be used in surface dressings done with bitumen emulsions.

To decide on the nominal size of chippings to be used, both the traffic intensity on the road and the hardness of the substrate should be considered. Larger sizes of chippings are an advantage for heavy traffic. On very hard substrates such as concrete or hard, weathered asphalts in which the chippings will not be embedded, smaller sized chippings are desirable. The interaction of these two variables to indicate the appropriate nominal size of chippings is shown in Table 7.1 abstracted from Overseas Road Note No. 3.

Experience will be necessary to classify a road surface appropriately amongst the five categories of hardness listed in Table 7.1. In acquiring this experience it is useful to employ a simple penetration cone test to measure the hardness of the road surface. An appropriate test has been developed using a modified soil assessment cone penetrometer. This test is also described in Overseas Road Note No. 3.

TABLE 7.1
RECOMMENDED NOMINAL SIZE OF CHIPPINGS (MILLIMETRES) (FROM OVERSEAS ROAD NOTE NO 3)

Type of surface	Approximate number of commercial vehicles with an unladen weight greater than 1.5 tonnes currently carried per day in the lane under consideration				
	2000-4000	*1000-2000*	*200-1000*	*20-200*	*Less than 20*
Very hard	10	10	6	6	6
Hard	14	14	10	6	6
Normal	20+	14	10	10	6
Soft	*	20+	14	14	10
Very soft	*	*	20+	14	10

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

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

* Unsuitable for surface dressing.

With double surface dressings, two different sizes of chippings are used with the intention that the smaller should pack into the voids between the larger. It has been a matter of debate as to which size of chippings should be used in the first dressing. In general it is best to apply the dressing with the larger sized chippings first so that the smaller chippings can be packed into the interstices between the larger chippings. However, there are two circumstances when it may be better to use the smaller chippings in the first dressing. One is when treating a very hard substrate to which larger chippings will not adhere. The other is in new construction when traffic will be allowed to run on the first dressing before the second dressing is applied. Some of the bitumen is likely to be absorbed into the surface of the new base, leaving insufficient to hold larger chippings in place.

7.2.3 State of old road surface

For surface dressings to succeed, the substrate must have a uniform surface texture. Any variations in the surface texture are likely to affect the performance of the dressing. The binder will drain down into areas that are open-textured whilst remaining on the surface where the texture is close and compact. On existing roads, potholes should be repaired in anticipation of surface dressing using patching materials which simulate the texture of the adjacent surfacing. Also, traffic should run over the newly patched surface for a month or so before the surface dressing is applied. Frequently, the existing surface is smooth and well compacted in the traffic lanes and more open and absorbent between the lanes. Some modern binder sprayers can cope with this situation using adjustments available on the spray bar to make a small reduction in the amount of binder sprayed over the traffic lanes.

Road surfaces should always be cleaned to remove any mud or other extraneous material before surface dressing is done and should be dry when the surface dressings are applied.

7.2.4 Rate of spread of binder

Specifying and obtaining the appropriate rate of spread of binder in an even film over the road surface is the most important factor in ensuring the success of surface dressings. With too much binder, it will flood up over the chippings producing a smooth slippery surface. With too little binder, the chippings will be dislodged under traffic.

Hansen (1934), in New Zealand, was the first to define a numerate method of determining appropriate rates of spread for different circumstances. He determined that in a loose layer of chippings spread evenly over a smooth surface, the voids in the layer are initially about 50 per cent. Under compaction with a roller, the voids are reduced to 30 per cent. Under further compaction by traffic, they are reduced still further to 20 per cent or less. For best results, between 50 and 70 per cent of the compacted voids should be filled with binder. Stone chippings are irregular in shape and they settle in place on the road with their least dimension vertical. Hansen introduced the concept of Average Least Dimension. This is, in effect, the average thickness of a single layer of chippings when they have been packed down into their final interlocking position. This concept is illustrated in Fig. 7.2. It is the Average Least Dimension rather than the nominal size of chippings which determines the appropriate amount of binder to be used.

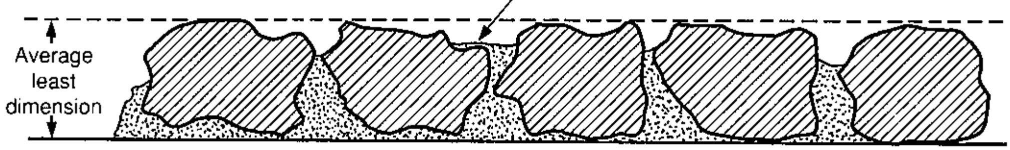


Fig.7.2 Use of Average Least Dimension to determine the rate of spread of bitumen.

The Average Least Dimension (ALD) is a function of the nominal size of the chippings and their characteristic shape. It can be determined by calliper measurements on a representative sample of about 200 chippings but another convenient way is to use the chart shown in Fig. 7.3 in association with the median size of the chippings and their Flakiness Index. The median size is the sieve size through which 50 per cent of the chippings pass. This is usually the mean of the passing and retained standard sieves by which the nominal size of the chippings is defined, with adjustments as necessary according to the proportions of oversize and undersize chippings found by grading analysis of the sample.

The rate of application of binder determined by the ALD method requires adjustment to take account of four factors; the traffic intensity, the characteristics of the existing road surface, the climate, and the nature of the chippings. Jackson (1963) derived numerical factors to define the effects of these four variables. In designing surface dressings, the factors for the particular road are determined and added together to give an overall weighting factor. The effects of this overall weighting factor are illustrated in the chart in Fig. 7.4.

Appropriate rates of spread of cutback bitumen can be determined from this chart. The intercept between the horizontal ALD line and the appropriate factor line indicates the required rate of spread of bitumen on the scale at the bottom of the chart. When penetration grade bitumens or bitumen emulsions are used, further adjustments are necessary. With penetration grade bitumens the indicated rate of spread should be reduced by 10 per cent to allow for the reduction in volume of cutback bitumens as the cutting-back oils evaporate. With bitumen emulsions the indicated rate of spread should be multiplied by (90/b) where b is the bitumen content (per cent by weight) of the emulsion. This factor allows for the water content of the emulsions.

Fig. 7.4 can also be used as a guide to the appropriate rate of spread of chippings to give uniform coverage, with the usual 10 per cent extra to allow for chipping losses. This rate of spread is determined by the intersection of the horizontal ALD line and the diagonal line labelled AB in the chart and is shown on the scale at the top. Some adjustment from this indicated rate of spread may be found to be necessary on site.

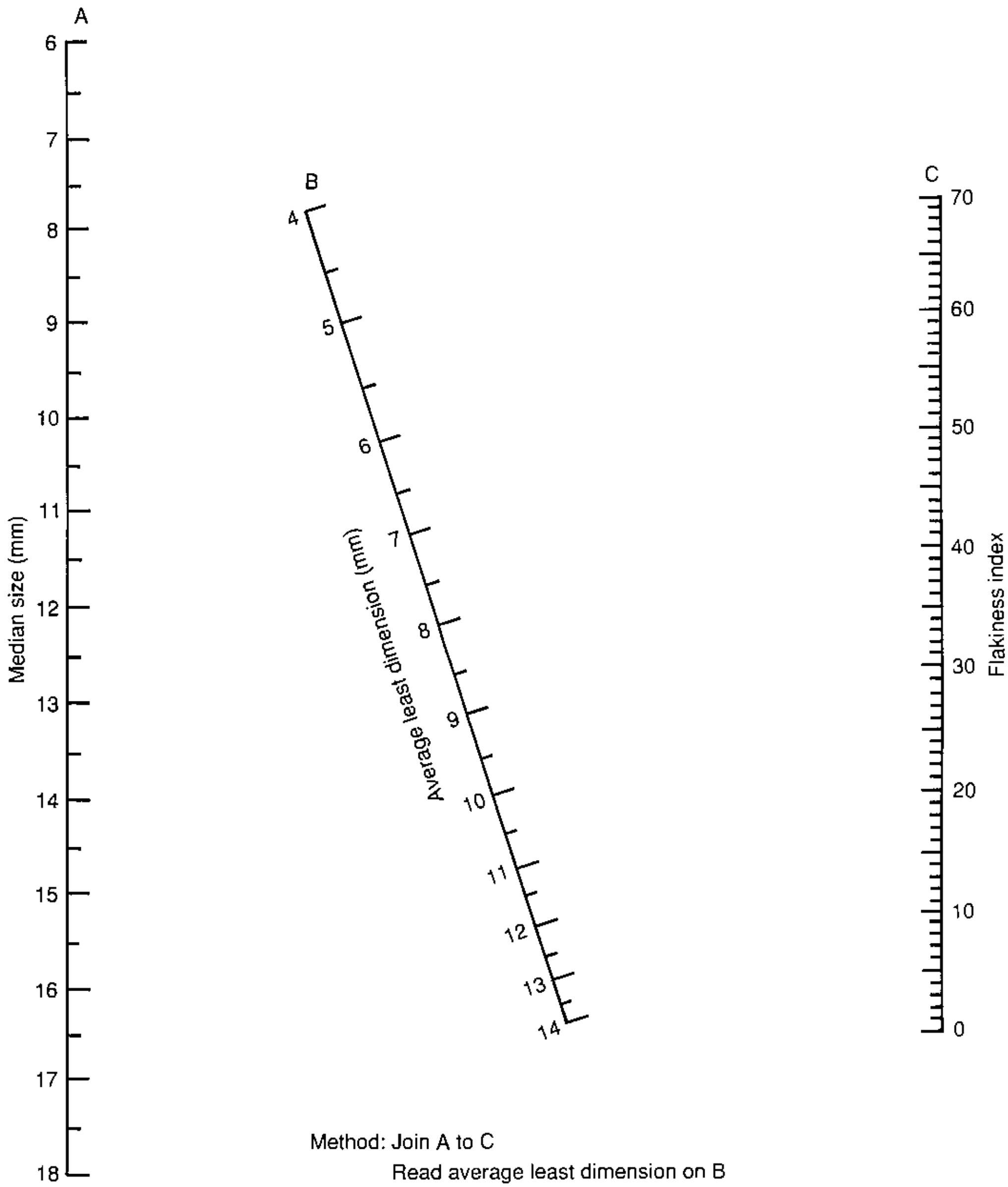
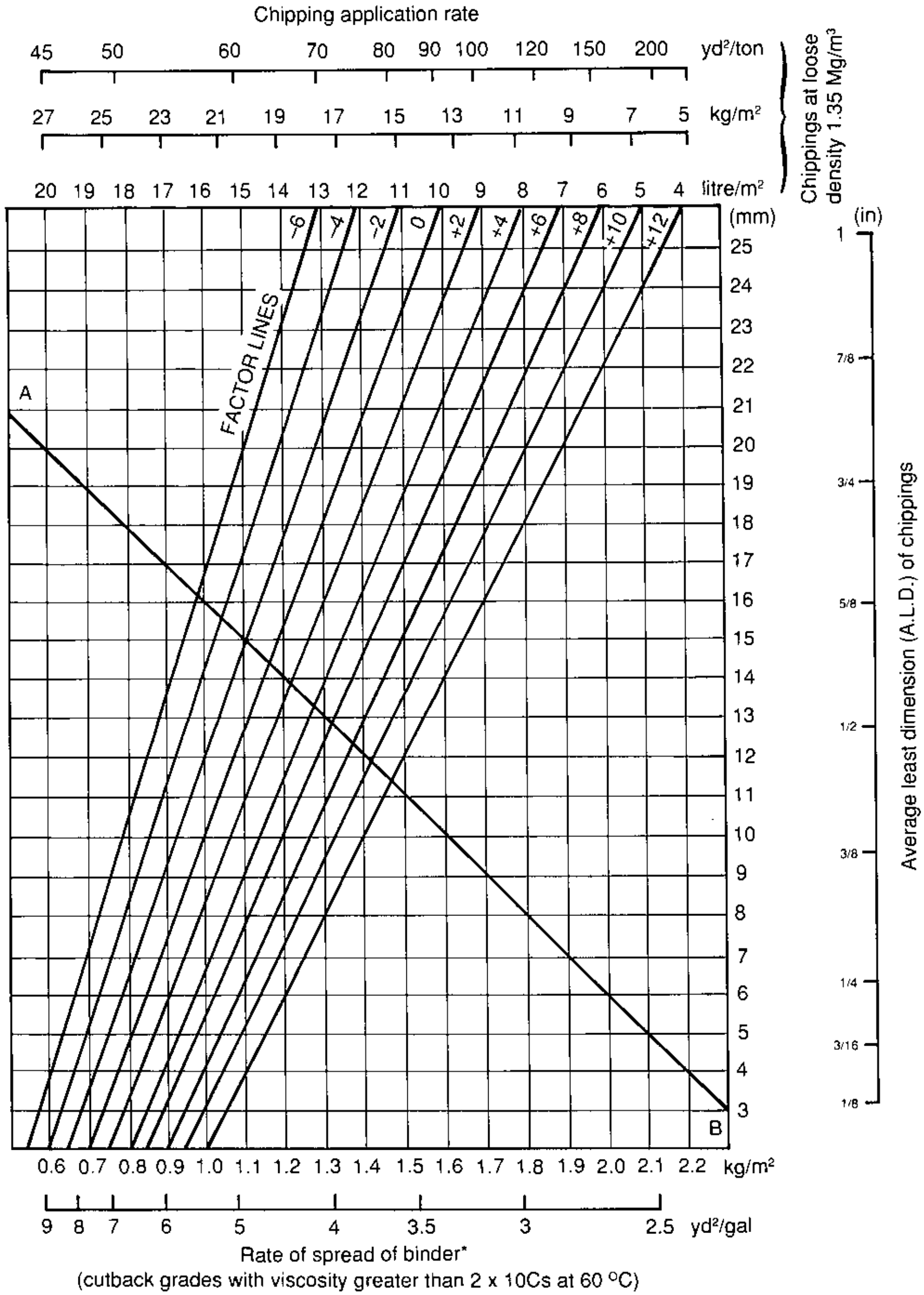


Fig.7.3 *One method of determining the Average Least Dimension of chippings.*



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.7.4 *Surface dressing design chart.*

7.3 Effects of wet weather

If rain falls on a bitumen emulsion before the emulsion has broken, the bitumen will be washed away to the side of the road. If rain falls on a newly laid surface dressing made with hot bitumen, it is possible that the water will displace the unhardened bitumen. The danger is greater with cutbacks than with penetration grade bitumens since it may take some hours for the bitumen to harden by the evaporation of cutting oils. These dangers are of some consequence in the variable Atlantic climate of western Europe but in the more predictable climates of many tropical countries it is often possible to confine surface dressing to periods of the year when the risk of rain is small.

With bitumen emulsions the cationic and anionic compounds used as emulsifying agents promote active adhesion with the aggregates and, once adhesion has developed between the bitumen and the chippings, water will not break the bond. With hot-applied bitumens, their high viscosity at ambient temperature is sufficient to resist the displacing action of water. Once the bitumen has hardened in the compacted surface dressing, there is no further risk of damage from rain. The danger period is within the first few hours after laying and the risk of damage is increased if fast moving traffic is allowed to run on the surface during this period.

In climates where there is a risk of rain falling on newly laid dressings, adhesion agents can be of value. Nowadays such agents are usually long-chain amines with polar properties similar to those used in the manufacture of cationic emulsions. Their effectiveness varies, being greatest with siliceous aggregates i.e. with aggregates which are most at risk from the superior wetting properties of water.

The effectiveness of particular agents and the proportions required can be determined from the Immersion Tray Test (see Chapter 5 and Overseas Road Note No. 3). These adhesion agents can degrade and lose their efficacy when held at bitumen spraying temperatures for more than an hour or so. Agents that are susceptible to such damage should be added to the bitumen in the binder distributor and mixed with the bitumen through the circulatory pumping system immediately before spraying.

Another method of using the agents, which also assists in dealing with dusty chippings, is to precoat the chippings with kerosene containing the adhesion agent. When belt conveyors are used to load chippings into the spreader, kerosene solution can be sprayed onto the chippings as they move up the belt. Alternatively, simple concrete mixers can be used, in both cases with precautions to eliminate the risk of fire.

7.4 Laying of surface dressings

Surface dressings can be laid by hand, the minimum mechanical equipment needed being a steel-wheeled roller to make sure that the chippings are well compacted into the binder. In some parts of the world it is done with quite primitive equipment e.g. with the binder being heated in an oil drum over an open fire and being applied to the road using watering cans or gallon tins with holes punched in the bottom. Such dressings, though of variable quality, may be the only possible

Fig.7.5 Surface dressing by hand.

means in self-help schemes to pave village streets. Bitumen emulsion makes such work easier since it needs no heating and the emulsion can be squeegeed over the surface to produce a more even and economical rate of spread. Small binder distributors are available with hand lances to spread the bitumen and gangs regularly employed in such work can acquire a high level of proficiency (see Fig. 7.5).

The output of such gangs is low, usually being less than 1000 square metres per day. The need for surface dressing in all countries is sufficient to merit the use of mechanised surface dressing trains capable of outputs of over 30,000 square metres per day. Such trains have three components, the binder distributor, chipping spreaders, and compaction equipment together with trucks to carry men and materials. There may also be hot storage tanks for the binder unless the work is sufficiently close to a depot from which hot bitumen can be transported to refill the binder distributor whenever this is needed.

7.4.1 Binder distributors

These have gone through several stages of evolution over the last 50 years. The earliest were pressurised tank distributors. In these, an air compressor is used to generate a constant high air pressure in the tank thereby delivering the binder at constant pressure to the spray bar at the rear of the machine. When it is necessary to adjust the width of spray, keys mounted in the spray bar can be turned on and off, either all together or individually, at either end of the bar. Operating under constant pressure, these jets each deliver a constant volume of binder in unit time and the rate of spread of binder is controlled by the forward speed of the vehicle.

To overcome the need for careful speed control, constant rate-of-spread distributors were developed. In these, binder is delivered to the spray bar through a metering pump driven from the main transmission of the vehicle through a multi-ratio gear box. The settings on this gear box can be adjusted from the driver's cab so that the appropriate volume of binder per unit time is delivered to the spray bar to give the specified rate of spread regardless of the forward speed of the vehicle. A disadvantage is that adjustments to the width of spray cannot be made whilst the machine is in motion. Such distributors were never put into large scale production. On all other binder distributors the primary means of obtaining the specified rate of spread of binder is by controlling the forward motion of the unit at the appropriate speed. Binder distributors have suitable gear ratios to facilitate speed control when moving slowly and a conspicuous speedometer is fitted in the driver's cab. Nevertheless errors do occur. It must be hoped that the future will bring a robust and versatile machine on which the rate of spread of binder does not depend on the vigilance of the driver in controlling the speed of the distributor.

Modern binder distributors are of two types, one working by constant volume and the other by constant pressure. In constant volume machines the binder is delivered to the spray bar through positive displacement pumps, the output of which can be adjusted. Spray width can be adjusted by adding extensions to the spray bar with appropriate adjustments to the pumps. Minor adjustments can be made to the spray width by opening and closing individual jets but such adjustments should be limited because they affect the overall rate of spread of binder. Some constant volume machines have facilities to circulate the binder through the spray bar to bring it to spraying temperature before beginning work. The control of the rate of spread of binder is not simple because it involves interdependent adjustments of the displacement pump and the driving speed. Most binder distributors made in the USA are constant volume machines and they are probably the most common type found in developing countries.

Constant pressure machines employ pumps to generate adequate pressure and this pressure is controlled by a pressure relief valve mounted in the spray bar assembly which permits binder to bypass the spray bar and return to the tank. In this way constant pressure is maintained in the spray bar. Unlike earlier machines, the tank itself is not pressurised. Spray width can be adjusted by adding extensions and by operating individual jets without any adjustment being needed to the pumping system. The rate of spread of binder is controlled by the forward speed of the machine. These machines have recirculating spray bars through which binder can be circulated to bring the bar to operating temperature. A typical machine of this type is illustrated in Fig. 7.6. Most binder distributors made in Great Britain are of this type. They are easier to operate to give a controlled rate of spread than the constant volume machines. One reason for this lies in the jets used to spray the binder.

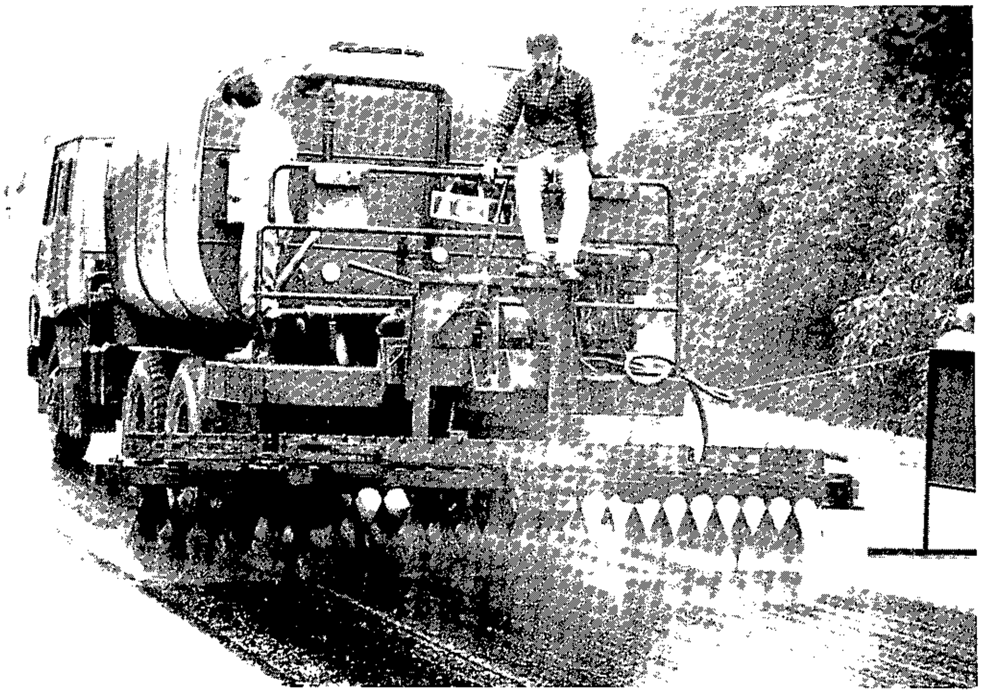


Fig.7.6 A constant pressure bitumen distributor.

7.4.2 Spray jets

There are two quite different forms of spray jet. One is the slotted jet which produces a fan-shaped spray. Such jets are set obliquely in the spray bar so that the sprays of fluid binder do not impinge on each other (see Fig. 7.7). The other is the whirling-spray jet. These consist of hollow cylinders into which the hot binder enters tangentially and is discharged as a hollow conical spray of binder droplets which pass through the cones of spray from adjacent jets.

Slotted jets are usually fitted to machines made in the USA. They have a rapid output and are particularly useful when heavy applications of binder are needed as in grouting and mix-in-place operations. Whirling-spray jets are normally used on distributors spraying hot binder in the United Kingdom. They have a lower output than slotted jets. This is an advantage in surface dressing, particularly when lighter rates of spread are needed for small chippings. At the lower forward speeds used with these jets, it is easier to control the rate of spread and to keep the chipping spreaders in pace with the spraying. However, they need more careful maintenance than slotted jets.

The British Standards describing the essential characteristics of binder distributors are:

BS 1707 (1989). Hot binder distributors for road surface dressing.
BS 3136, Part 2 (1972). Cold emulsion spraying machines for roads.

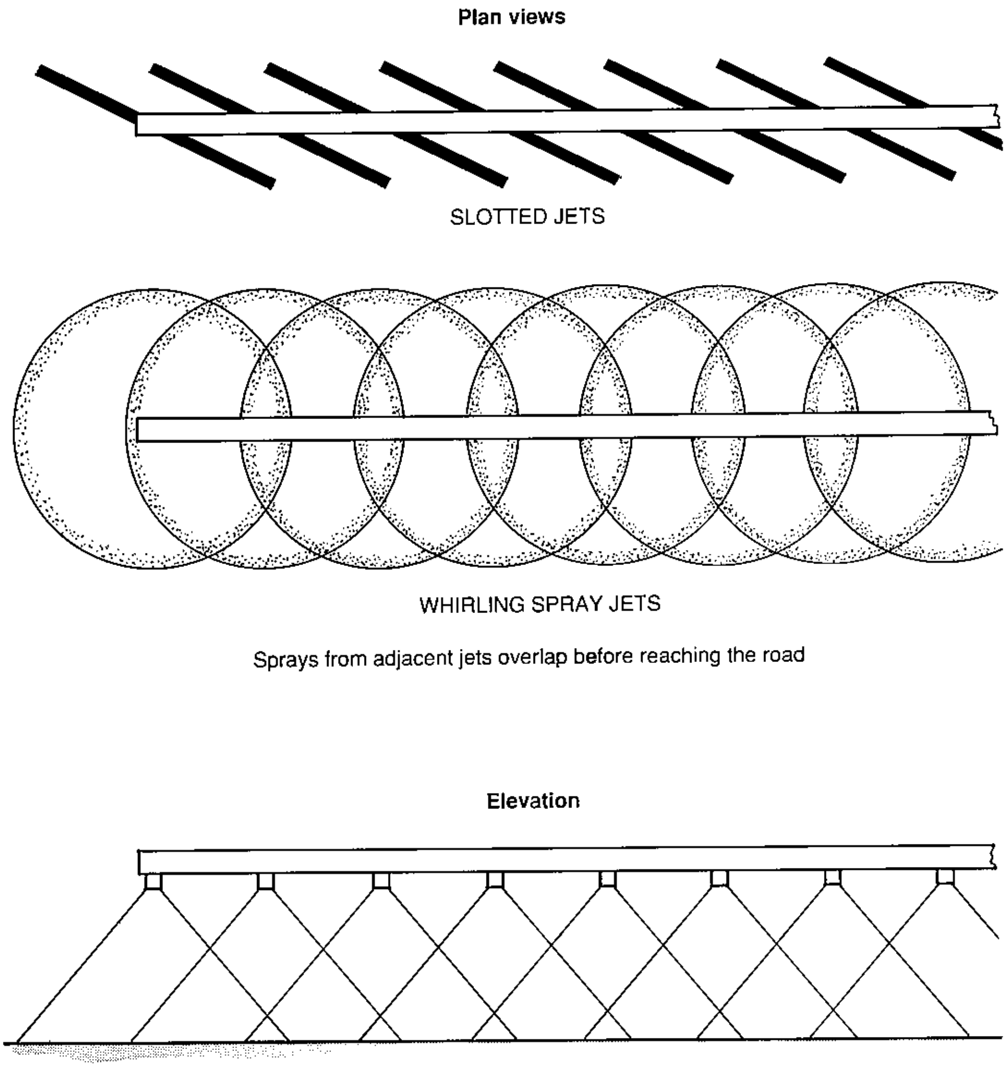


Fig.7.7 Form of spray from slotted and whirling spray jets.

With both types of jet it is an essential feature that the spray bar be kept high enough above the road for the spray from one jet to intermingle on the road with the spray from the two adjacent jets on either side, as shown in Fig. 7.7. This is necessary to produce an even rate of spread across the road. Spray bars with whirling-spray jets are protected with a hood to prevent wind from disturbing the spray. Spray bars with slotted jets are usually not protected in this way and this may explain why operators frequently run their distributors with the spray bar too close to the ground, resulting in the typical longitudinal striations which mark many defective surface dressings, as shown in Fig 7.8.

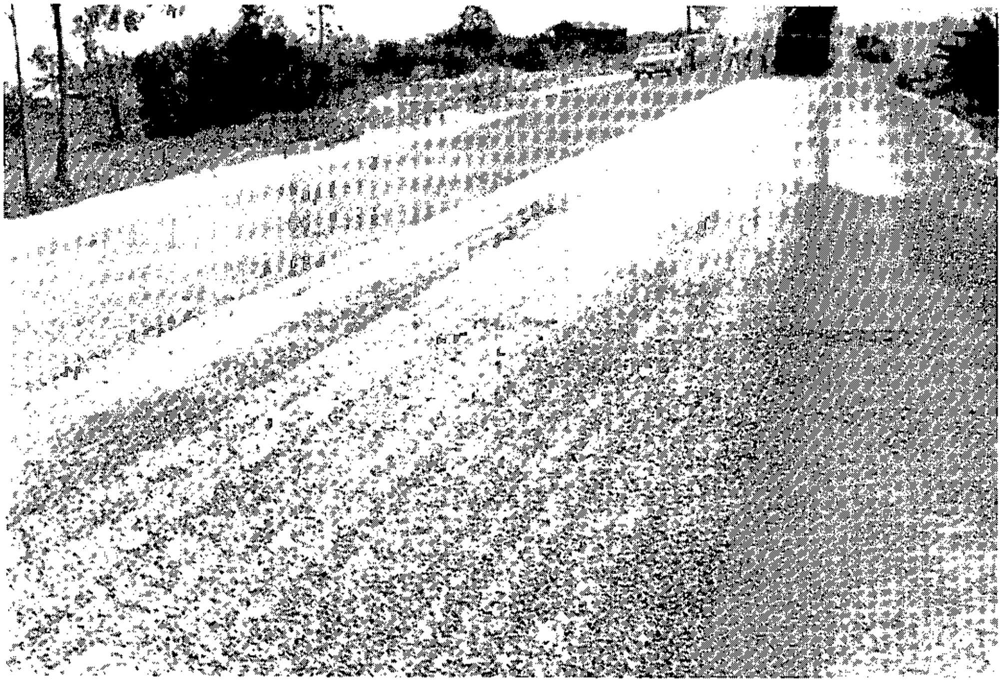


Fig.7.8 Longitudinal striations produced by a spray bar which is set too low.

Binder distributors should be checked regularly to establish that the spray bar is delivering a uniform lateral spread of binder. The Depot Tray is employed for this purpose and, in any country where surface dressing is done, there should be facilities at some central depot where the distributors can be brought at regular intervals for testing. The test is illustrated in Fig. 7.9. In this test, the binder distributor is set up with the spray bar over a series of metal troughs, each 50 millimetres wide. The spray bar is operated until the troughs are nearly full. The transverse distribution of binder so determined is then plotted as shown in Fig. 7.10. BS 1707 describes this test and prescribes limits of ±15 per cent on permitted variations from the mean. Good surface dressing contractors will seek to make sure that the transverse distribution on their machines varies by no more than ±10 per cent from the mean.

To check how well the specified rate of spread of binder is achieved, an overall estimate can be obtained by using the dip-stick on the distributor before and after a particular measured area has been completed. In addition, the rate of spread can be measured at different points by the use of weighed metal trays of known area, usually about 0.1 square metre. Four or five of these are placed in the path of the sprayer and the weight of binder sprayed on each is determined.

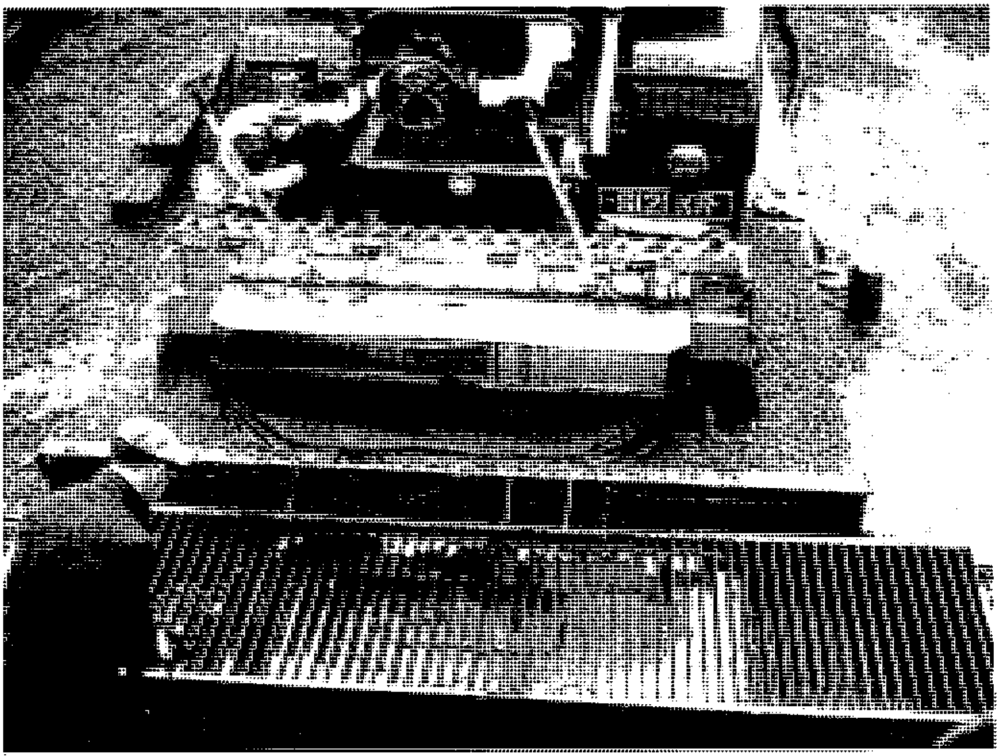
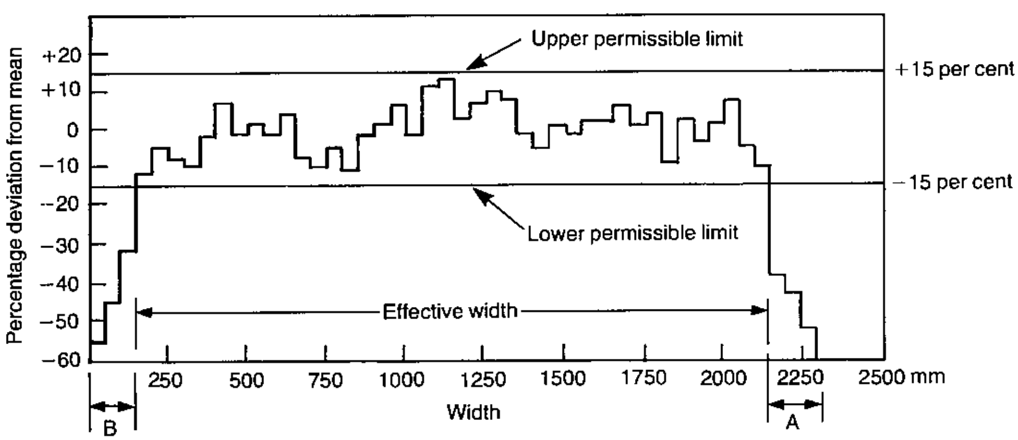


Fig.7.9 *The Depot Tray Test for distributor spray bars.*



Note: The binder falling on widths A and B is ignored in calculating the mean distribution.

Fig.7.10 *Results of a typical Depot Tray Test for the transverse distribution of bitumen.*

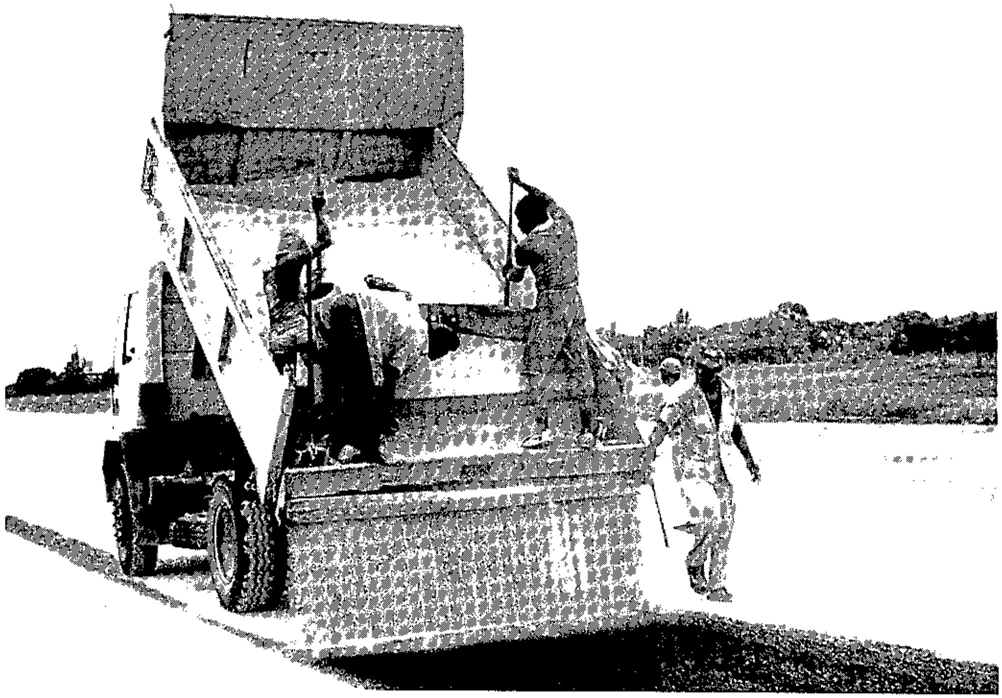


Fig.7.11 A tail-board chipping spreader.

7.4.3 Chipping spreaders

The simplest chipping spreaders are tailgate attachments fitted to the rear of the trucks used to haul the chippings to the site. These are quite inexpensive attachments and their use does not reduce the payload of the trucks (see Fig. 7.11). The simplest is probably the Hornsey gritter. With this device the flow of chippings is controlled by an operator who walks alongside the tailgate as the tipping lorry reverses. An elaboration which dispenses with the need for the walking operator uses a crimped roller to meter the chippings. This roller is driven by an attachment to the road wheels, thus ensuring that a constant rate of spread of chippings is maintained even when the speed of the truck varies. A further elaboration involves the use of a wheeled hopper with a metering roller which is pushed by the reversing truck as it tips its load into the hopper.

Drivers can acquire considerable skill in reversing their trucks and simultaneously operating the tipping mechanism. Uniform rates of spread of chippings can be obtained with drivers who have acquired this skill but the most effective means of spreading chippings are the self-propelled metering chip spreaders. There is now a wide selection of such machines available, all working on the same principle. They consist of a hopper at the rear, into which tipping trucks discharge their load, and a conveyor which carries the chippings forward to the distribution hopper. At the bottom of this there is a metering roller which passes an even flow of chippings on to the road.

The driver has a clear view of the road ahead and is able to operate the controls necessary to make sure that the flow of chippings is regulated to the rate of cover desired.

7.4.4 Rollers

Rolling is important to make sure that the chippings are well bedded into place. Steel-wheeled rollers are traditional but tend to crush weaker aggregates. They should not exceed 8 tonnes in total weight. Pneumatic-tyred rollers are preferred since their rolling action manoeuvres the chippings to pack down into a close mosaic and they do not bridge over slight depressions in the road surface. For high quality work, rubber-shod steel-wheeled rollers are available. Rollers should follow closely behind the chipping spreaders to help in establishing good adhesion whilst the binder is still warm. Slow moving traffic can help in building up this adhesion and, provided speeds can be restricted to less than 25 km/hr, roads can be opened to traffic immediately after rolling is completed. Fast moving traffic can damage newly laid dressings, particularly during rapid acceleration, turning and braking, therefore roads should not be opened to unrestricted traffic until it is quite evident that the dressing has been well compacted into place.

7.4.5 Aftercare

In order to obtain a complete cover of chippings it is usual to apply a slight excess, with the result that there are inevitably loose chippings on and beyond the edges of newly completed dressings. These may be removed by hand brooming but mechanical brooms are more effective and they can also find use in cleaning the surface before work is started. Manhole covers and traffic studs marking traffic lanes can be covered with adhesive sheet before spraying and one of the aftercare tasks is to remove these adhesive sheets and make sure that the appliances are in good working order.

7.5 Special forms of surface dressing

There are many variants of normal surface dressing. In South Africa, for example, precoated grit is sometimes spread over finished surface dressings to hold the chippings more firmly in place. This is a variant of the double surface dressings used to surface newly constructed roads. In France there are recent developments in the use of bitumen emulsions. In one, for instance, designed to extend the surface-dressing season, a layer of uncoated chippings, normally of 20 millimetres nominal size, is spread one stone thick over the road surface. This is then sprayed with bitumen emulsion. The surface is then blinded with smaller chippings and then rolled with rubber-tyred rollers. In Norway good results have been reported from the use of graded aggregates i.e. chippings containing a substantial amount of undersize fines. These can all be regarded as attempts to produce a thin layer of aggregate-binder mixture on the road surface. It may turn out that such methods are a means of reducing the care necessary in the design and execution of conventional surface dressings, a mitigation that may be helpful in Third World countries. But no clear conclusions have yet emerged on how well their durability compares with conventional dressings.

In some parts of the world where there are difficulties in obtaining chippings of crushed rock or gravel, sand may be used for surface dressing the more lightly trafficked roads. Sand seals are less durable than conventional surface dressings. The sand is abraded away by the action of traffic and the mixture of sand and bitumen that remains provides less protection against weathering and the entry of water than the thicker film of binder used in conventional surface dressings. Both cutbacks and emulsions can be used in such dressings, usually at a rate of spread of approximately 1 kg/m^2, and the sand should be clean and as single sized as possible. Recommendations for sandseals have been published by the National Institute for Road Research in South Africa.

7.5.1 Slurry seals

These are, in effect, elaborations of sand seals. Fine-graded aggregate is mixed with bitumen emulsion at ambient temperatures to produce a slurry which can be spread over the road surface in a thin layer sufficient to fill the surface voids. Emulsions with relatively high bitumen content are used, often with small proportions of cement to stiffen the mixture. Such slurrys may be used to treat old roads or airfields where the surfacing has a 'hungry' appearance. They are useful in restoring a uniform surface texture to old roads that are to be surface dressed. Slurry seals may also be used on newly laid single surface dressings as an additional means of holding large chippings in place and also as a means of treating surface dressings that, through either incorrect design or defective application, have developed surface faults. Recommendations for the design and application of slurry seals have been published by Hawken (1967) and by the Asphalt Institute. They may be prepared in concrete mixers at the side of the road but the most convenient and effective means of treating large areas is by using mobile mixer-spreaders. The manner in which these machines operate is illustrated in Fig. 7.12. They can apply up to 8000 square metres of slurry seal per day.

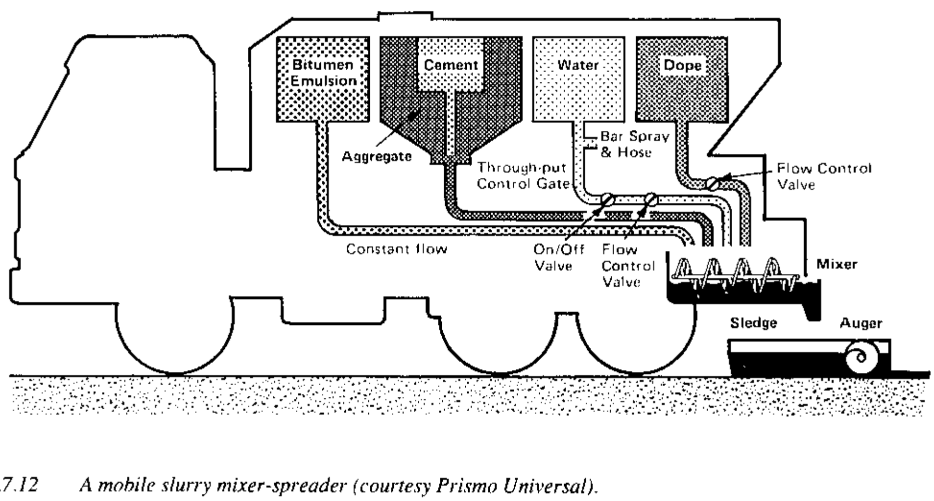


Fig.7.12 A mobile slurry mixer-spreader (courtesy Prismo Universal).

The predominant purpose of slurry seals is cosmetic, to produce a smooth even appearance on roads which have developed an irregular surface texture. Any effect they may have in sealing the surface against the entry of water is only temporary. Cracks in the road surface are likely to reappear quite quickly through the slurry seal. Slurry seals should not be regarded as substitutes for surface dressing, either in durability or in sealing surfacings against the entry of water.

7.5.2 Calcined bauxite-epoxy resin surface treatments

The calcining of bauxite ore at high temperatures produces a highly abrasive and strong aggregate which can be used to make surfacings with an extraordinarily high resistance to skidding. Produced in Guyana, this aggregate can be used in normal surface dressings but its relatively high cost justifies its use for special extra durable surface treatments which use binders made from mixtures of epoxy resin and bitumen. These treatments are surface dressings in which the epoxy resin-bitumen mixture is used to glue the calcined bauxite to the surfacing below. Such treatments can be used on concrete and steel bridge decks. They have found their main use at busy junctions in towns and cities where, as reported by Hatherley (1978), their use has produced quite remarkable reductions in accident rates. Specialist contractors have the skills required in formulating and laying the bitumen-epoxy resin mixtures for different circumstances.

7.6 Present needs

In some developing countries, surface dressing is not used at all. In others it is undertaken as part of major road building projects but the expertise is not available to be drawn on when needed for road maintenance. In some countries surface dressing is done by hand with varying degrees of success. Very few countries have the mechanised surface dressing trains for this process which is so effective in preserving the investment in bituminous surfaced roads. Its neglect is one reason for the sad story in the World Bank's policy study 'Road Deterioration in Developing Countries'. What can be done to remedy this situation?

Part of the difficulty lies with the systems currently being advocated to plan the programmes of road maintenance and rehabilitation in developing countries. Behind them lies the laudable principle that all road works should have a sound and demonstrable economic justification. Benefit-cost studies are now in use all over the world to determine road building priorities and standards. In Third World countries the most demonstrable benefits derive from eliminating the heavy costs of operating vehicles over rough and bumpy road surfaces. Such benefits are used to determine the traffic levels at which it is appropriate to up-grade roads from a gravel to a bituminous surface. They are also appropriate in determining priorities in programmes to rehabilitate roads that have fallen into disrepair. But for planning road maintenance strategies, these systems have a serious flaw. The chief criterion they use for initiating action is high levels of road surface roughness. A ready economic justification can be found for remedial works which reduce the inflated operating costs of vehicles using bumpy roads. What a waste of resources is thereby involved. The upper layers of the pavement are allowed to disintegrate so that much of the original investment is lost. Traffic has to bear the inflated costs of operating over the pot-holed surfaces for several years before the resources can be mustered to replace the upper layers of the pavement. All of these represent heavy costs for the fragile economies of developing countries to bear.

Road pavements are designed so that, with proper maintenance, they will last for 15-20 years before strengthening and possibly widening are needed to cope with the increasing traffic. Even then, much of the value of the pavement is preserved. Indeed, there is evidence that in hot countries and with effective maintenance, road pavements can become stronger with the passing years (see Chapter 9, Pavement Design). There should be no need for these expensive intervals in the life of a pavement when the upper layers are allowed to collapse, increasing vehicle operating costs and incurring the heavy cost of pavement renewal.

Because of the rather rapid weathering of the bitumen in premixed surfacings which occurs in hot climates, a surface dressing may be required earlier than in more temperate climates. Indeed, it is not uncommon for surface cracking to develop in the first five years. Neglected, these cracks can lead to early pavement failure. If a surface dressing is applied when needed, the pavements will continue to give good service until the time comes for them to be strengthened and widened to cope with increasing traffic. Clearly, in the systems of planning road maintenance there should be a component to indicate the need for surface maintenance *before* the pavement collapse occurs. Surface maintenance will frequently involve only the relatively cheap process of surface dressing. An important and necessary step in promoting the greater use of this process in developing countries lies in revising the systems of assessing road maintenance needs so that the value of preventative maintenance is more heavily emphasised than at present.

What other motivation is required for countries to develop their capacity to employ this most effective process of road maintenance? Surface dressing requires a considerable investment to provide the mechanised trains that are needed on the scale required. A high level of technical skill is also needed in both design and execution. In Australia it was the State Highway Departments that were motivated to develop their own surface dressing trains in the course of providing and maintaining all-weather, dust-free surfaces on the great length of rural road. They have developed the process to a high level of technical refinement. In Europe, much of the original motivation came from the suppliers of alternative binders, road tar, bitumen and bitumen emulsions, keen to sell their products. Realising that they would profit by selling a service, they were responsible for the initial development of effective binder spraying equipment. Their ability to supply binder, accurately sprayed in the required amounts, was eagerly accepted by highway authorities. Nowadays, in Europe and America almost all surface dressing is done by specialist contractors.

In developing countries it may not be easy to build up a new contracting industry with specialist skills and a sizeable investment in machinery. One suggestion is that bitumen suppliers be encouraged to provide their bitumen on site in bitumen sprayers together with the expert staff to operate them. Another is to provide technical assistance to develop skills in surface dressing within highway departments and to permit, even encourage, the leaders who emerge to set up their own contracting organisations or to join existing local contractors who see the advantages of adding surface dressing to the services they provide.

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8 Roadbases and sub-bases

This chapter is primarily concerned with flexible pavements. The information on sub-bases is also relevant to the design and construction of concrete pavements. Sub-bases are almost invariably natural soils of low plasticity. On more heavily trafficked roads the upper layer of the sub-base may be made with cement or lime-stabilised soil. Bases fall into two main groups, unbound and hound. Unbound bases are those which rely on their intrinsic internal friction to develop the necessary bearing capacity i.e. crushed rock and natural gravels. Bound bases are those in which a binder (cement, lime or bitumen) is used to enhance their ability to reduce the traffic stresses on the layers of pavement below.

8.1 Crushed stone bases

8.1.1 Dry-bound and water-bound macadam

Telford construction i.e. the use of roughly trimmed boulders as illustrated in Fig. 4.8, is still used in parts of the world where supplies of crushed stone are not readily available. But it is more effective to employ a little extra effort to break stones down to particles no larger than 50 millimetres in size. These are used to make the dry-bound and water-bound macadams which derive directly from the precepts of John Loudon Macadam. In these two processes, roadbases are built up in layers, each layer being about 100 millimetres in compacted thickness. In dry-bound macadam, stone of 40-50 millimetre size is spread over the substrate. This is then blinded with aggregate to fill the voids and the whole is then compacted with a heavy steel-wheeled roller. Dry-bound macadam is used in areas where water is not readily available and, for this, it is an advantage to use vibrating rollers. In water-bound macadam the fines are slurried into place with copious applications of water during compaction, with more fines being added as necessary to fill the voids. In hot dry weather, continuing the compaction as the water evaporates can produce bases of quite high compacted densities.

The fines should be non-plastic and contain no particles larger than 5 millimetres in size. Breaking stone by hand, or with rudimentary crushing equipment, does not produce fines in the quantities required and it is usually necessary to seek out supplies of natural sandy soils. This form of construction was widely used in the Indian sub-continent and is still used in more remote areas. The sandy soils used, ubiquitously known as 'murram', usually contain quite large proportions of clay. Though this is an advantage in binding these materials when they are used as running surfaces, it can be a source of weakness when the time comes to provide such roads with bituminous surfacings.

8.1.2 Graded crushed stone

With the advent of effective quarrying equipment, graded crushed stone bases have displaced the macadams in many parts of the world. They have advantages in speed and convenience of construction and, though more mechanical equipment is needed, it is easier to make sure that the final product is uniform and consistent with specified requirements. The aim is to use a mixture of stone of different sizes so that the grading approximates to a Fuller curve in the expectation that, when compacted, the material will be impermeable and the stiffness and resistance to further compaction under traffic will be maximised. Crusher-run rock, the material deriving from the final crushing operation in the quarry (see Fig. 4.9), is often used. Whenever it is particularly important to produce a high quality base, chippings of different sizes are recombined to produce mixtures of appropriate grading. Mechanical spreaders are used to lay the material, usually in layers of uncompacted thickness of less than 200 millimetres. Water must be added to assist in compaction and, in hot dry weather, continuing the compaction as the material dries out is an advantage in securing high levels of compaction. With crushed stone bases made with recombined single-sized chippings, water can be added as the chippings are remixed. This material, known as 'wet-mix', has the advantage that the presence of water inhibits segregation of the different sizes during transport and laying.

With both the macadams and the graded stone bases, there are four important considerations,

(a) to select an aggregate of adequate mechanical strength and free from suspicion that the aggregate minerals are partially weathered.

(b) to provide the aggregate on site in a suitable range of particle sizes.

(c) to spread the mix uniformly and compact it to the correct thickness and high density so that its flexural strength and resistance to further compaction under traffic are developed to the highest possible degree, and that its permeability is minimised.

(d) to produce a surface finish to the correct line and level and with a profile free from bumps and hollows within specified tolerance limits.

8.1.3 Mechanical strength and resistance to weathering

Methods of evaluating these properties have been reviewed in Chapter 4. Very high strength is not essential. The Ten Per Cent Fines Test (BS 812, Part 111) is probably the most appropriate method of evaluating mechanical strength and the effects of moisture on mechanical strength should also be investigated. Typical specifications are given in Table 8.1.

Where compression testing machines are not readily available, other simpler tests such as the Modified Aggregate Impact Test may be used in quality control testing (Hosking and Tubey (1969)). There is no unique relationship between the results of the different tests used to evaluate the mechanical strength of aggregates; correlation between the results of any two tests must be established for each individual aggregate.

Surface rocks in the tropics have often been weakened by partial weathering of the minerals they

TABLE 8.1
MECHANICAL STRENGTH OF AGGREGATES FOR ROADBASES AS DEFINED BY THE TEN PER CENT FINES TEST

*Climates**	*Annual rainfall (mm)*	*Minimum 10% fines values (kN)*	*Minimum ratio wet/dry test results (%)*
(a) Humid and monsoon	> 250	110	75
(b) Arid and semi-arid	< 250	110	60

* These climates correspond to the environmental categories defined for pavement design in Chapter 9. Categories 1 and 2: humid and monsoon. Category 3: arid and semi-arid.

contain and aggregates used in roadbases should be free of this defect. Methods of preventing the use of such weathered rock are outlined in Chapter 4.

8.1.4 Particle size and shape

For dry-bound and water-bound macadams the coarse stone particles may be produced by hand labour or they may be produced from mechanical crushers. They will usually be transported to the site and roughly spread by tipping trucks. Final spreading can be done using hand rakes but nowadays this will usually be done with a blade-grader or small bulldozer. The objective should be to use cubical particles, rejecting any that are flaky or elongated and aiming for a uniform size with the maximum dimension not exceeding 50 millimetres. For contractural purposes it will usually be necessary to quantify these requirements and typical grading limits are given in Table 8.2.

The most suitable fines for use with dry-bound and water-bound macadam are undoubtedly the crushed rock fines remaining after the final quarry screening of aggregates. These will normally be of 5 millimetres maximum size and graded down to dust. It will usually be necessary to

TABLE 8.2
COARSE AGGREGATE FOR DRY-BOUND AND WATER-BOUND MACADAM

Sieve size (mm)	*Percentage passing nominal sieve size*	
75	100	100
50	85-100	85-100
37.5	0-30	35-70
28	0-5	0-15

Note: The first of these gradings corresponds to the 50 mm nominal single sized material as specified in BS 812 and is appropriate for use with mechanically crushed aggregate. The second is a broader specification suitable for use with hand-broken stone. In both, the product should conform to the limits on flakiness and elongation such as prescribed in BS 812.

TABLE 8.3
GRADED CRUSHED STONE BASES

BS test sieve (mm)	*Percentage by mass of total aggregate passing test sieve*		
Nominal size (mm)	37.5 mm	28 mm	20 mm
50	100	-	-
37.5	95-100	100	-
28.0	-	-	100
20.0	60-80	70-85	90-100
10.0	40-60	50-65	60-75
5.0	25-40	35-55	40-60
2.36	15-30	25-40	30-45
0.60	8-22	13-27	15-30
0.075	5-12	5-12	5-12

augment this with supplies of natural fine aggregate. A sharp concreting sand is an appropriate addition. Silty or clayey sands should not be used when the macadam is a base under a bituminous surfacing.

Typical grading specifications for graded crushed stone bases are given in Table 8.3.

One of the difficulties in using graded crushed stone lies in avoiding segregation of the different particle sizes during transport and laying. Segregation can be reduced by keeping the aggregate damp and by limiting the maximum size of aggregate to no more than 37.5 millimetres.

The amount and properties of the fines fraction can have a considerable influence on the behaviour of graded crushed stone bases. Preferably, the fines should be non-plastic. A maximum Plasticity Index of 6 in the fines is often specified. This is appropriate with material in which the fines content is near the lower limits of the grading but the presence of clay becomes more deleterious when the fines content approaches the upper limit. A more appropriate method of controlling this effect is to require that the Plasticity Product (PP) should not exceed 60, where,

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

8.1.5 Spreading and compaction

Graded stone bases are most effectively spread with the spreading and finishing machines used for laying bituminous mixtures. These control segregation and facilitate working to correct line and level. With skilled operators, spreader boxes and even blade-graders can be used. With blade-graders it is desirable to restrict the maximum size of aggregate to 36.5 millimetres or lower in order to make it possible to grade the finished surface accurately. Graded stone bases are normally laid in uncompacted thicknesses of no more than 200 millimetres in the expectation that this thickness will be reduced by 25-30 per cent during subsequent compaction.

Appropriate compaction levels are normally specified in terms of Relative Density and it is usual to aim for densities between 98 and 100 per cent of optimum in the BS Heavy Compaction Test or the equivalent Modified AASHTO Compaction Test. The standard laboratory compaction tests are done on samples from which all particles over 20 millimetres in size have been removed. Thus some care must be used in deriving Relative Densities from measurements of the field densities of crushed stone which contains particles up to 50 millimetres in size. Direct use of the densities derived from the laboratory tests can result in a large overestimate of the state of compaction of the crushed stone base. The densities measured in the laboratory tests should be adjusted to allow for the proportion of material over 20 millimetres in the compacted base and its specific gravity. Checks can be made on the validity of this adjustment by carrying out laboratory compaction tests on samples in which the oversize material is replaced by stone particles between 10 and 20 millimetres size.

CBR testing is not appropriate in evaluating crushed stone bases. The need to remove particles of over 20 millimetres size from the test samples is a disabling limitation and, in any case, the CBR values derived from well compacted crushed stone are outside the range of applicability of the test.

Good compaction is essential on roads carrying the overladen vehicles common in some developing countries. The loads applied to the road through the rear wheels of grossly overloaded vehicles are of the same order as the loads applied by compaction equipment. It is therefore important to be particularly vigilant in compacting unbound bases to such a level that further compaction in the wheel tracks of heavily laden vehicles is reduced to a tolerable minimum.

Vibrating steel-wheeled rollers are the most effective in compacting crushed stone bases. On large contracts it is desirable to carry out compaction trials with the compaction equipment available to determine which is the most effective and the number of passes required (and to obtain other equipment if none of those first offered proves effective).

The intrinsic variation in field density measurements of crushed stone bases is notoriously high. In the United Kingdom this variability has prompted the use of method specifications for compaction in which the type of equipment, the number of passes and the moisture content of the material are specified (further details are given in Chapter 11). In developing countries it is usually necessary to persevere with field density measurements as the final arbiter on whether adequate levels of compaction have been achieved. With the guidance obtained from preliminary compaction trials and from any previous experience in the area with the same materials, it is possible to provide the contractor with guidance on how the specified levels of compaction can be consistently achieved. Nuclear densimeters are useful in monitoring compaction levels. They require much less effort and time than any direct method of measuring field density and it is possible that, as more experience is gained, they will come into use as the primary means of checking conformity with compaction specifications for crushed rock and for other road making materials.

8.1.6 Line and level

In urban areas it is usually important to control road surface levels accurately in relation to adjacent footways and buildings. On rural roads the precise level of the finished surface is less critical. The important considerations are that each layer of the pavement is constructed to the specified thickness and that the finished surface is smooth with the appropriate camber or crossfall so that rainwater is discharged rapidly into the side drains. On roads in developing countries, the initial bituminous surfacing is usually quite thin, being either a surface dressing or a bituminous premix which is often only 50 millimetres thick. This means that the surface of the compacted roadbase must be finished to a high level of accuracy. A straight edge can be used to check for local irregularities and an appropriate specification is that there should be no deviations in the finished surface of the base of more than ±5 millimetres under a 3-metre straight edge. Camber, crossfall and overall longitudinal profile can be checked by levelling.

With crushed stone bases containing particles of 50 millimetres size, it is not easy to work to close tolerances in surface finish and there are advantages in using material with a smaller maximum nominal size in the uppermost layer of the base. There is no easy way of correcting a faulty surface profile in a crushed stone base. This is one reason for the traditional use of two-course bituminous surfacings in Europe, the base course being used to correct inequalities in the profile of the substrate before laying the more expensive wearing course.

It is important to carry out regular checks to determine that the thicknesses of both base and sub-base are as specified. An investigation in East Africa by O'Reilly (1974) on roads with a specified thickness of base of 150 millimetres showed that actual thicknesses varied between 75 millimetres and 180 millimetres. More recently, Rolt *et al* (1987) have shown that deviations of ±50 millimetres from design thicknesses for granular layers are very common.

8.2 Natural gravel bases

8.2.1 Lateritic and quartzitic gravels

These materials occur in many tropical and sub-tropical countries. They are used in making and maintaining gravel roads and suitably selected gravels can also be used to make bases for bituminous surfaced roads. Their origins, mode of occurrence and properties are described in Chapter 3. In some countries, notably in Kenya and Malawi, accessible deposits of these materials are becoming scarce. When used in making gravel roads, they are eventually lost by the combined erosion of traffic and weather, and attempts are now being made to reserve them for use as bases in bituminous surfaced roads where their value will be preserved as part of a permanent structure.

Texturally they are clayey gravels and their suitability as roadbases is a function of their particle size distribution and their clay content. In some deposits the material is good enough to make bases for all but the most heavily trafficked roads. Other deposits contain material suitable for use as sub-bases but which can be stabilised with cement or lime to manufacture roadbases. They often have a natural particle size distribution that is close to that required for high stability and they are considerably easier to manipulate than crushed stone.

The coarse particles in quartzitic gravels are usually adequate in mechanical strength. In lateritic gravels the nodules are generally very hard and tough and they have high specific gravities reflecting the presence of the concretionary iron oxides which bind them together. In some

TABLE 8.4
RECOMMENDED GRADINGS FOR NATURAL GRAVEL BASES

BS test sieve (mm)	*Percentage by mass of total aggregate passing test sieve*		
Nominal size (mm)	37.5mm	20mm	10mm
50	100	-	-
37.5	80-100	100	-
20.0	60-80	80-100	100
10.0	45-65	55-80	80-100
5.0	30-50	40-60	50-70
2.36	20-40	30-50	35-50
1.18	17-37	-	-
0.60	12-27	15-30	15-35
0.30	9-24	-	-
0.075	5-15	5-15	5-15

Note: Not less than 10 per cent should be retained between each pair of successive sieves except the two largest sizes.

deposits, laterisation has not proceeded very far and, although the nodules may be present in profusion, their mechanical strength may be low. As is shown in Fig. 3.3, there is an approximate correlation between the specific gravity of lateritic nodules and their mechanical strength. The presence of weakly laterised nodules can usually be recognised by hand-inspection. The effects of weak nodules will usually be revealed by untypically low values in the CBR tests used to assess their suitability. In surveying the potential of the gravels in available deposits, the same methods of evaluating the mechanical strength as are used for crushed stone are appropriate (see Section 8.1.3). Typical specifications for particle size distribution are given in Table 8.4.

These specifications reveal a trend to push the gradings towards the continuous Fuller curve which was derived by Fuller and Thompson (1907) in seeking to define ideal aggregate gradings for producing dense concrete. It has never been conclusively proved that this necessarily produces the most suitable materials for road making, either in asphalt surfacings or in unbound bases. The 'plums in the pudding' type of mixture can be equally satisfactory as shown by the performance of dry-bound and water-bound macadams. It may even have advantages, as in stone-filled sand asphalts (Chapter 6). Lateritic gravels are often gap-graded with an absence of material between the 10 and 2.36 millimetre sieves, as illustrated in Fig. 3.2, but they should not be rejected on this account.

The fines in both quartzitic and lateritic gravels usually contain some clay. In wet and seasonally wet climates it is usual to limit the plasticity of the fines to a Plasticity Index of not more than 6. In arid and semi-arid climates this limit can be increased to 12. As with graded crushed rock, the level of tolerable plasticity increases with decreasing fines content and the Plasticity Product, as defined in Section 8.1.4, may be used to allow for this.

CBR testing is used to evaluate these materials for use as bases and sub-bases. This testing should be done on samples that have been compacted to the density anticipated under compaction in the field. Good field compaction is necessary with these materials to muster their full bearing capacity, to minimise further compaction in the wheel tracks of heavy vehicles and to make them less susceptible to the weakening effect of any water that may enter them. Reasonable target values for field compaction are 97 per cent or higher of the maximum density achieved in the BS Heavy Compaction Test or the US equivalent, the Modified AASHTO Compaction Test.

In arid or semi-arid climates it will be appropriate to derive CBR values from tests on samples compacted to 97 per cent of this optimum level and at the corresponding optimum moisture content. In wet and seasonally wet climates it may be expected that water will enter the base through cracks in the surfacing at some stage. For this reason the tests should be done on samples that have been immersed in water for four days. CBR values of 80 per cent or more are indicative of material suitable for use as a roadbase. Gravels with lower CBRs are likely to be useful for sub-bases and to have potential for use as bases when stabilised with cement or lime.

Care is necessary in extracting these gravels to avoid contamination from underlying clayey soils. The floor of deposits is usually horizontal and obvious to the eye. Extraction from a vertical face is often an advantage in mixing together material from different horizontal bands.

These gravels are easy to spread using bulldozers and blade-graders. As with graded crushed stone, they should be compacted in layers not exceeding 200 millimetres in uncompacted thickness. Rubber-tyred rollers and steel-wheeled rollers are suitable and vibration is an advantage in securing the high levels of compaction desirable. These gravels are easier to manipulate to correct line and level than crushed stone and they have the additional advantage that when irregularities are evident in the finished surface it is not difficult to scarify and recompact the top 100 millimetres or so of material.

8.2.2 Other gravels

River gravels are often suitable for making roadbases if they are first crushed so that the particles are angular and can develop the internal friction on which the stability of unbound bases depends. Such crushing may produce insufficient fines and it may therefore be necessary to add non-plastic fines to make mixtures with stable gradings such as those defined in Tables 8.2 and 8.3. Where river gravels and sands are readily available it may be worth considering a concrete pavement.

Physical weathering of rocks produces fragmentation, often resulting in deposits in which the particles can range from boulder size down to fine aggregate. These materials are often found in the foot-slopes of mountains and may be sorted further by ground movements to produce terrace gravels. A danger with such materials is that chemical weathering of the minerals they contain may be quite advanced, though not evident in casual hand-inspection. The origins and mode of occurrence of these materials are reviewed in Chapter 3. The danger from chemical weathering and measures to avoid using weathered material are the same as those employed for evaluating potential quarry sites in basic rocks as described in Chapter 4. In other respects these materials can be treated in the same way as other gravels for making roadbases. Grid rollers and sheepsfoot rollers can be useful in the initial compaction of these gravels to break down the small boulders they are likely to contain. The fines may be found to be excessively plastic and, should this be so, it is worth considering the use of cement or lime stabilisation to reduce their susceptibility to the weakening effects of water.

8.2.3 Corals and soft limestones

These materials can be used to make quite strong roadbases. Their mechanical strength is likely to vary over the area of the deposits. They are often used as fill material to raise road foundations above local flood levels and this gives an opportunity to locate sources of harder material for making roadbases. These materials are usually too soft to be put through normal quarry crushing and grading procedures and they are used 'as dug' with care during compaction to preserve their somewhat fragile structure. Beaven (1971) has produced a useful review of information on the uses of coral and soft limestones in road building.

8.3 Soil stabilisation

Many soils can be stabilised with cement, lime or bitumen. Stabilised soils are one form of bound base, the purpose of the stabiliser being to supply a cohesion which is lacking in unbound bases. When used in low proportions, the primary purpose of cement and lime is to react with the clay fraction in the soil to reduce the susceptibility of the compacted soil to the weakening effect of increasing moisture content. There is a whole spectrum of materials ranging from these cement-modified soils through cement-bound granular bases and lean concrete to the structural concrete used in making roads and buildings. Lime requires the presence of clay in the soil and cannot be used to bind non-plastic soils. The binding action of cement and lime results from complex chemical reactions with the soil in the presence of water. In contrast, the stabilising action of bitumen is entirely physical, the bitumen acting as an adhesive to hold the aggregate particles together. Again, there is a spectrum of materials ranging from bitumen-stabilised sands through grouted macadam bases and premixed macadam bases to the asphalts used for surfacing flexible pavements. The use of bitumen is confined to non-plastic soils and aggregates since any clay that is present absorbs much of the bitumen to no purpose.

There are other stabilising agents. In the past, some have been offered commercially as water-proofing agents for soils e.g. aniline furfurol, but none has proved as effective, permanent and cheap as cement, lime and bitumen. Others, such as sulphite lye and calcium chloride, have hygroscopic properties and can be useful in arid climates in retaining moisture in earth and gravel roads, supplying some cohesion, and reducing dust emissions. It is sometimes claimed that this enhanced cohesion is also of value with non-plastic bases in arid climates. Surface-active agents have been used to economise on the water needed for compaction in arid areas and it is likely that freshly applied hygroscopic materials could have a similar action but they have no other purpose as stabilisers for roadbases.

Soil stabilisation can produce very durable road pavements. In Zambia, for instance, many main roads constructed with lime-stabilised gravels in the 1950's remain in serviceable condition despite lack of maintenance. In northern Nigeria there is an outstanding example of a bitumen-stabilised sand pavement on the Maiduguri-Bama road which was constructed in 1960 and which was still in serviceable condition in the late 1970's.

The World Bank's report, 'Road Deterioration in Developing Countries' (1988), lists a series of policy recommendations aimed at arresting this deterioration. One aspect of engineering policy to this end lies in the use of methods of road construction that produce robust pavements which can survive with a minimum of maintenance. Soil stabilisation offers a practical means of securing this objective, both in the construction of new roads and in the rehabilitation of roads in which the pavements have deteriorated to the extent that they need to be reconstructed.

8.4 Cement and lime stabilisation

8.4.1 The binding action of cement and lime

The cement used is normally Portland cement. The lime may be quick lime or hydrated lime i.e. either the oxides of calcium and magnesium, CaO and MgO, or the hydroxides $Ca(OH)_2$ or $Ca(OH)_2$ MgO. Cement is available in all countries. Lime may have to be specially imported but where calcareous rock is readily available it can be worthwhile encouraging a cottage industry to produce burnt lime for road making as was done in Ghana and Tanzania. Methods of producing burnt lime in small kilns have been described by Ellis (1974).

Since Roman times, lime has been mixed with natural pozzolans (e.g. volcanic ash) to produce cementing mortars. In industrialised countries, the ash from coal-fired electricity generating stations is providing a valuable source of pozzolan. This pulverised fuel ash (PFA) is used as an admixture in commercially supplied cements and is also mixed with lime in soil stabilisation. In tropical countries where sugar cane and rice are grown, the vegetable wastes, bagasse and rice husks, are burnt. The resultant ash contains substantial amounts of amorphous silica which is an active pozzolan and which can be mixed with burnt lime for use in soil stabilisation (see Cook and Suwanvitaya (1982) and also Mehta (1979)). These mixtures of lime and pozzolans produced by burning, develop quite early strengths resembling those produced by cement (Portland cement is a pulverised mixture of burnt lime and burnt clay).

Most lime stabilisation is done with mixtures of lime and natural soil without the addition of such pozzolans. The cementing properties derive from reactions between the lime and clay minerals in the soil. When lime is added to clayey soil, calcium ions replace the sodium ions in the clay until it becomes saturated with calcium and the pH rises to more than 12. The lime required to complete this replacement is termed the initial consumption of lime and may be determined by testing (see BS 1924 (1990), Methods of test for stabilised materials for engineering purposes). Above pH 12 there is a dramatic increase in the solubility of the silica and alumina in the soil and reactions proceed to form the calcium silicates and aluminates which are the cementitious materials which bind the soil together. The conversion from sodium to calcium ions produces a marked reduction in the apparent plasticity of the soil. With gravelly soils this may be sufficient to transform a gravel of marginal quality into a material quite suitable for use as a base. On mixing the lime with the soil, this reaction is immediate. The gain in strength associated with the formation of calcium silicates and aluminates occurs more slowly and may continue for many years. It is accelerated by heat, an advantage when using lime stabilisation in hot climates.

The primary use for cement and lime stabilisation in tropical countries has so far been with gravelly soils to produce roadbases. The processes can also be used with more clayey soils to make the upper layer of sub-bases. Often, the soil subgrade along the line of the road can be used in this way provided it can be broken down to the fine tilth necessary for mixing in the stabiliser. Recent experience in both the USA and the United Kingdom has shown that soils in which sulphates are present should be avoided. In both countries, examples have been reported of lime-stabilised clays swelling to a marked degree in the months following construction. The cause of this swelling has been traced to a reaction between sulphates in the soil and the calcium silica-alumina hydrates formed as the lime reacts with the soil. This reaction produces the minerals ettringite and thaumasite which can absorb large quantities of moisture, swelling as they do so (see Mitchell and Dermatas (1990) and Snedker and Temporal (1990)). This reaction can occur in the presence of as little as 0.3 per cent of sulphate in the soil and is reported to be activated in situations where the soil is in or near a saturated condition.

Quick lime is denser than hydrated lime and can be produced in a range of aggregate sizes. It is less dusty than hydrated lime. Both are skin and lung irritants and it is advisable for operators to wear protective clothing and face masks, particularly when handling the more toxic quick lime. Quick lime is particularly useful in stabilising soils that are very wet. As soon as it contacts the wet soil it hydrates and absorbs a large amount of water. The reaction is exothermic and the heat produced assists in drying the soil further.

Some ion exchange occurs in cement stabilisation but the initial consumption of cement in this exchange is not yet fully understood (see Ballantine and Rossouw (1989)). In hydrating, the cement forms calcium silicates and hydrates of alumina which produce a bonded structure in the soil. It also releases lime that reacts with any clay present to produce further bonding as outlined above.

This description and the notes that follow on design and construction are necessarily rather brief. Amongst the references at the end of this chapter are included publications dealing specifically and in detail with cement and lime stabilisation.

8.4.2 Design of cement and lime-stabilised soils

Design testing is required for three purposes. Preliminary testing can be used to eliminate soils that are intrinsically unsuitable for stabilisation. Further testing is then necessary to determine the proportion of stabiliser required and to establish target levels for field compaction. This further testing will also reveal the structural properties to be assigned to the material for pavement design purposes.

In preliminary testing, particle size analysis will be sufficient for some decisions. Non-plastic soils are not suitable for stabilising with lime but many can be stabilised with cement. The most suitable are those that are evenly graded i.e. with gradings approaching those used for unbound bases defined in Table 8.4. Gap-graded soils are also suitable provided there are sufficient fines to fill the voids between the coarse particles. Desert sands are usually not suitable. Even if stable gradings can be found, or made by mixing different sands, the resultant sand-cements tend to be friable and to break down under traffic. Such sands can be more effectively stabilised with bitumen.

Preliminary testing will also include the preparation of trial mixes to eliminate any soils which are contaminated with material inimical to the soil-stabiliser reaction. One such is organic material. Although organic material from decayed vegetation does not persist in the soil for very long in tropical climates, it may occur in surface soils where vegetation has only recently been cleared. Any sulphates present, either in the soil to be stabilised or in the underlying sub-base or subgrade, are likely to be harmful. The chlorides often present in soils in a marine environment are less damaging unless present to excess. Should montmorillonite be present amongst the clay minerals, its large volume changes with changing moisture content may disrupt the stabilised soil. The testing of soils to quantify the presence of these materials is an unnecessary complication. Their presence can be inferred if trial mixes fail to harden in the expected manner. It may be interesting, but not immediately useful, to pursue the reasons for any failure to harden. Design testing should continue only on those soils with which there is demonstrable hardening of the trial mixtures.

Gravels in which the coarse particles derive from basic rocks which may be partially weathered, should not be used. Although they may show an initial gain in strength when stabilised, the weathered particles are likely to decompose further, leading to disruption of the material.

With lime stabilisation the only preliminary tests required involve measuring the changes in plasticity of the fine fraction of the soil. These changes begin immediately on mixing and are likely to increase for a long period. Mixtures containing more lime than is needed for the initial consumption are eventually likely to become completely non-plastic. The preliminary samples of lime-stabilised soil should be kept damp for 7 days and if the PI has then decreased by 50 per cent or more, further testing is warranted. If these tests are done on samples containing, say, 3 per cent and 6 per cent of lime, they will also give a preliminary indication of the effects of lime content.

Cement-stabilised soils harden quite rapidly, particularly at high ambient temperatures. After 7 days curing they are likely to have reached about half their ultimate compressive strength. The gain in strength of lime-stabilised soils is slower and continues for much longer. The methods of test must take account of this difference in hardening characteristics. Cement-stabilised soils are usually evaluated by unconfined compression tests but CBR testing is employed in evaluating lime-stabilised soils. In the methods of test prescribed in BS 1924 the test specimens are cured in damp conditions for 7 days and then immersed in water for a further 7 days before testing. Tests are done with a range of stabiliser contents, typically 2, 4, 6 and 8 per cent by weight. Typical design criteria are given in Table 8.5.

TABLE 8.5
STRENGTH CRITERIA FOR STABILISED SOIL BASES AND SUB-BASES

Uses	Cement stabilised	Lime stabilised
	Unconfined compression strength (1) (MN/m^2)	Minimum CBR value (1) (%)
Base Type A (2)	3.0 to 6.0	100
Base Type B (3)	1.5 to 3.0	80
Sub-base	0.75 to 1.5	40

Notes. (1) Both after 7 days curing + 7 days immersion in water as in the test methods detailed in BS 1924.
(2) Type A for use on more heavily trafficked roads.
(3) Type B for use on more lightly trafficked roads.

At this stage, decisions are dominated by considerations of how the whole pavement is to be designed and it is necessary to anticipate matters dealt with more fully in Chapter 9. All stabilised soils tend to crack because of shrinkage of the mixtures during hydration of the stabilisers. Such cracking can be controlled within reasonable limits by proper curing during construction but it must be recognised that cement and lime stabilisation do not produce a continuous monolithic layer (Bofinger *et al.* (1978)). With lime stabilisation and with low proportions of cement, these cracks are likely to be small and closely spaced. The material is best regarded as a series of closely interlocking, irregularly shaped pieces. In other words, for pavement design purposes, stabilised soils can be regarded as a form of unbound base. Higher cement contents produce tensile strengths sufficient to inhibit this localised cracking and the strains induced by initial shrinkage and by subsequent temperature changes are relieved by the development of wider cracks which are further apart. These are often transverse cracks approximately 3 to 5 metres apart i.e. resembling the spacing of joints used in concrete roads to relieve temperature stresses.

How much do these cracks matter? With the thin bituminous surfacings used in developing countries, the larger cracks are likely to reflect through the surfacing quite quickly. Water entering through such cracks will not damage the base since stabilised soils are strong enough to resist the hydro-dynamic stresses that can damage saturated unbound bases under traffic. Cracks appearing in a surfacing may injure the pride of an engineer in his work but there are three circumstances when we can live with them:

(a) In arid areas where rainfall is negligible.

(b) When the lower layers of the pavement are permeable to the extent that water entering the cracks can be easily dispersed.

(c) On roads which are, of necessity, built only a metre or so above a water table. In this case, soil subgrades are likely to be saturated and a little more water will do no harm.

In other circumstances, water entering through cracks is likely to weaken the pavement. There are two methods of preventing such damage. On lightly trafficked roads the lime or cement content of the stabilised soil can be limited to a level at which the cracks are small and closely spaced, i.e. as in Base Type B in Table 8.5. On more heavily trafficked roads, sufficient thickness of pavement should be provided above the stabilised soil to prevent reflection cracking in the surfacing. For this purpose a minimum thickness of material of 150 millimetres is desirable. This material may be graded crushed stone with a thin bituminous surfacing or, on road pavements designed to carry very heavy traffic, a more durable method is to use a bituminous base and surfacing.

8.4.3 Carbonation

The concern about increasing levels of carbon dioxide in the atmosphere has spread to road engineering. If air has access to cement and lime-stabilised soils during the hardening process, the carbon dioxide contained in the air can combine with the stabiliser to produce calcium carbonate, reducing the pH of the material and inhibiting the hardening process (see Bagonza *et al* (1987), Paige-Green *et al.* (1990), de Wet and Taute (1985)). Carbonation is likely to occur if the stabilised soil is allowed to dry out during hardening, particularly if there are repeated cycles of drying and wetting.

Fig.8.1 *Mechanical spreader for cement or lime.*

8.4.4 Construction

It may sometimes be possible to stabilise the soil subgrade to provide material of adequate strength for a sub-base. More usually, both sub-base and base are made from material brought in from nearby deposits. It is spread loosely to the correct uncompacted depth and the required quantity of stabiliser is then spread over the surface and mixed in place with the soil, water being added as necessary during the mixing. The mixture is then compacted to correct line and level. Provision must then be made to keep the mixture damp for at least 7 days so that hardening proceeds satisfactorily. Careful site control is needed at all stages.

The stabiliser may be spread by hand. Bags of stabiliser are placed in a regular pattern over the surface to provide the correct amount of stabiliser. They are then split open and the stabiliser is raked evenly over the surface of the soil. Mechanical spreaders are more accurate. They are equipped with meters by which the required rate of spread can be obtained, regardless of their forward speed, and they can be fitted with skirts to reduce the dust nuisance. They are available as trailers and as self-propelled units. A typical spreader is illustrated in Fig. 8.1. Chipping spreaders used for surface dressing can be adapted for this use.

Mixing can be done with agricultural soil-tilling equipment i.e. with towed disc-harrows and spike-harrows. Their mixing efficiency is not high and to compensate for this it is usual to increase the dosage of stabiliser by 25-30 per cent. More effective mixing requires the use of specialised machinery. These range from multi-pass mixers, towed and powered by tractors, to heavy duty single-pass machines. Multi-pass machines can usually mix to a depth of 200 millimetres of uncompacted soil. Single-pass machines, such as that illustrated in Fig. 8.2, can mix to uncompacted depths of 350 millimetres. Thus, with multi-pass equipment, the thickness of one layer of finished compacted material should not exceed about 150 millimetres. With single-pass machines the limiting depth of one layer is likely to be determined by the efficiency of the compaction machines but it is not likely to be more than 200 millimetres.

When the required total thickness of stabilised soil is greater than the above limits, it should be laid in two layers. When this is done, great care is necessary to make sure that a layer of unstabilised soil is not left as unwelcome filling in a sandwich. Construction of the second layer should follow as quickly as possible on the first and the mixing machinery should be set so that it penetrates for at least 20 millimetres into the lower layer. In two-layer construction it is worth considering the use of plant mixing with plant set up in the quarry. The mixed material is then transported to the site to be laid with spreader boxes or more sophisticated spreaders and finishers. Pugmills similar to those used for making bituminous premixes must be used. Rotating and tilting drum mixers are not suitable.

Fig.8.2 Single pass mixer for cement or lime stabilisation.

8.4.5 Curing

Adequate arrangements for curing cement and lime-stabilised soils are vital for the stabilised soil to achieve its full potential as a load distributing component of the pavement. Proper curing is one of several methods of inhibiting cracking during the hardening process and it also prevents carbonation. Good curing involves nothing more than making sure that the mixtures are kept damp whilst the hardening process is active. The first 7 days are critical and it is beneficial, for lime-stabilisation particularly, if the material remains damp for some time thereafter.

Curing is usually easy in the cool, damp climate of northern Europe. It can be difficult in hot climates, especially in dry weather. On bases the bituminous prime coat is sometimes applied immediately after the stabilisation is completed in the belief that this will inhibit evaporation of water. This is a mistaken belief. The prime coat itself is permeable and its black colour increases surface temperatures under solar radiation and increases evaporation of water. There are two ways of obtaining effective curing. The first is by repeated applications of water during the curing period, assisted, where practicable, by a layer of sandy soil over the surface which can be removed (and reused) after curing is complete. The other method is to cover the surface with a light coloured impervious sheet such as polythene. Water, condensing from vapour on the underside of this sheet, falls back to maintain the moisture content of the soil. It is also an advantage to complete the construction of further layers of base or surfacing to cover the stabilised soil as soon as possible after the 7 day curing period.

8.5 Bitumen stabilisation

Bitumen stabilisation is most useful in areas where the climate is hot and dry. There are three reasons for this. First, sandy soils frequently predominate in these areas, indeed they may be the only readily available road making material. Second, water is not needed at any stage during the mixing and laying process. And third, in arid regions the bearing capacity of soils and sub-bases is often high so that quite thin roadbases are required, thus offsetting the relatively high cost of the binder.

Bitumen-stabilised bases always require a bituminous surfacing to prevent them from being abraded by traffic and to protect the binder from the natural weathering which is particularly active in hot dry climates. A sand seal may be used but a surface dressing with crushed stone chippings will be far more durable and protective because of the greater thickness of the binder film.

Local sands are likely to be dry or, at worst, only slightly damp and their natural temperature may be as high as 40°C. Cost and energy can therefore be saved by using them without any further heating. Mixtures can be made with cutback bitumens that are sufficiently fluid at ambient temperatures, such as MC30, but this complicates the laying process since the mixed materials must be left in a fluffed-up condition for the fluxes in the bitumen to evaporate before the material is compacted. There is, as yet, insufficient experience to report on the use of bitumen emulsions but the same limitations are likely to apply. With unheated sand it is better to use heated bitumen using the highest viscosity of cutback that is possible i.e. MC70 or MC250 grade. These contain less flux to be evaporated before the mixtures become sufficiently stable to carry traffic. But the best mixtures are probably made with a penetration grade bitumen of medium hardness e.g. 100 pen. With penetration grade bitumens it is necessary to heat the sand to temperatures of 120-160°C. The extra energy involved in this heating is comparable with the energy lost in the evaporation of the fluxes in the cutbacks.

The bitumen and sand should be mixed in a pugmill type mixer of the kind used in asphalt plants. The overall plant requirements are quite simple. In addition to trucks, the simplest consists of a small asphalt mixer, a small bitumen heater and a steel-wheeled roller, all other operations being done by hand. When greater outputs and a higher level of quality control are needed, further items of plant include loading hoppers and elevators, an aggregate drier, large asphalt mixers, bitumen heaters and a spreader-finisher for laying the material.

8.5.1 Design of mixtures

The sand should be well-graded, approximating to a Fuller curve. A typical target grading is shown in Fig. 8.3. In desert areas, sand deposits tend to be of single-sized material and it may be necessary to mix sands from deposits containing material with different particle size distributions to obtain a suitable mixture. The presence of small proportions of silt and clay is not a disadvantage. Where the sand is put through an aggregate drier, much of this fine material will be blown away as dust. The total amount of fines (passing the 0.075 millimetre sieve) in the final mixture should not exceed 12 per cent because their high surface area will unnecessarily increase the amount of bitumen needed to coat the material. The bitumen content will normally range between 4 and 5.5 per cent by weight of the total mix, the higher bitumen contents being appropriate with sands containing higher proportions of finer material. There is a close correlation between the amount of bitumen required and the total surface area of the sand particles. The Hveem Stabilometer is probably more suitable than the Marshall Test in indicating the influence of aggregate grading and bitumen content on the mechanical properties of these mixtures. In such testing, the fluxes in cutback bitumens must be allowed to evaporate before the samples of mixed materials are compacted in the testing moulds. Also, the mixtures should be compacted at a temperature approximating to that of the materials when they are compacted on the road. The stiffness of the mixtures on the road will increase as the binder hardens under the action of weathering.

Bitumen-sand mixtures can be adequately compacted with 6-10 tonne steel rollers and, although it is not necessary to test the field densities to which they are compacted, it is most necessary to carry out regular analysis tests to make sure that the grading of the sand and the bitumen content are kept within specification limits.

In some areas, sands with naturally stable gradings can be found. These can be used to make sand-asphalt surfacings, as has been done in southern Nigeria and in Umtali in Zimbabwe, and they can be used in the manufacture of asphaltic concrete and the stone-filled sand asphalt surfacings described in BS 594.

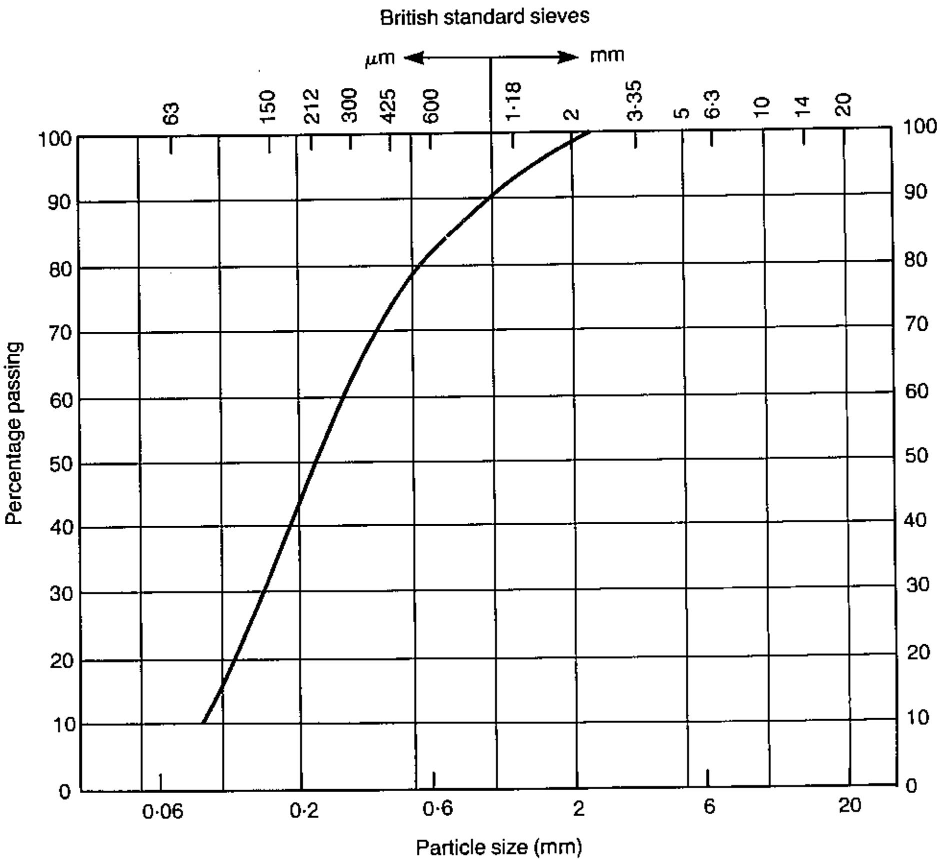

Fig.8.3 *A typical target particle size distribution for a sand suitable for stabilisation with bitumen.*

8.5.2 Bitumen-bound bases.

On very heavily trafficked roads it is desirable that the upper layers of flexible pavements be more substantial than is normally provided by present practice in many developing countries i.e. an asphalt surfacing 50 millimetres thick laid on an unbound base or a base of stabilised soil. In Europe and North America all new flexible pavements, except those to carry very light traffic, have asphalt surfacings at least 100 millimetres thick. On very heavily trafficked roads the bituminous surfacing and base often have a total thickness of more than 200 millimetres. Unlike unbound bases, these materials are able to provide tensile strength under dynamic loading. They therefore have a much greater capacity to reduce the traffic stresses in the lower layers of pavements and are advantageous, even necessary, for road pavements with weak foundations.

In asphalt wearing courses the maximum size of stone is usually less than 20 millimetres i.e. less than half the compacted thickness of the wearing course. Asphalt basecourses and roadbases are laid in greater thicknesses and mixtures with larger sizes of stone, up to 40 millimetres, can be used with advantage. They are made with penetration grade bitumens, usually of the same grade as is used in the surfacing. Research is in progress to establish how much the dynamic stiffness of these mixtures can be enhanced still further by the use of modified bitumens. This is an indication of the importance given to dynamic stiffness in providing durable flexible pavements for very heavily trafficked roads.

Asphalt basecourses and roadbases are defined by means of recipe specifications i.e. by the grading of the aggregate, the bitumen content and the state to which it must be compacted. Two types of asphalt base are used in the United Kingdom, hot rolled asphalt, with detailed specifications in BS 594, Parts 1 and 2, and dense coated macadam, with detailed specifications in BS 4987, Parts 1 and 2, as described in Chapter 6.

Because of the large particles of stone that these mixtures contain, it is not possible to evaluate them using the Marshall Test or any other established simple testing procedure. Even if it were possible, the results would not give a reliable indication of the desirable properties of those materials.

The British Standard specifications were originally evolved from experience. Over the last 20 years they have been refined by a combination of laboratory studies and road experiments. In these experiments on heavily trafficked roads, pavements were constructed with different forms of base, each in a range of thicknesses. Some of these experiments were laid in the late 1950's and early 1960's and their performance has been monitored over the last 30 years. The results have provided a major source of data for the recently published review of the design and performance of pavements by Croney and Croney (1991). There have been parallel laboratory studies of the mechanical properties of asphalt bases which included their elastic moduli, their fatigue strength and their deformation characteristics. Comparison with the results of the road experiments has clarified the significance of the laboratory measurements of the mechanical properties. In broad terms, the results emerging from both the road experiments and the laboratory tests show that the hot rolled asphalt bases are superior in performance to the dense coated macadam bases but they are also more expensive, so some judgement is involved in deciding which to use.

These experiments again illustrate the value of controlled road experiments to evaluate the relative performance of different forms of construction with local materials and in local climatic environments. The Overseas Unit of the Transport Research Laboratory is cooperating with Highways Departments in several countries in planning, constructing and monitoring the results of such experiments.

The apparatus used in the laboratory testing is complex and expensive and the testing methods are time consuming. In their present form they cannot be used in routine design testing. Efforts continue to develop methods of design testing for asphalt mixtures that are simple, accurate and indicative of field performance. This is one of the main objectives of the concentrated programme of road research initiated by the Federal Government of the USA, the Strategic Highway Research Program (SHRP).

Further information on bitumen-bound granular materials can be found in Chapter 6.

8.6 Cement-bound granular bases and lean concrete

These materials cannot be regarded as legitimate components of flexible pavements since they are intended to perform as rigid monolithic layers in the pavement. They are an example of a general trend to combine two different road building techniques in the hope of obtaining the advantages of both and the demerits of neither. Cement-bound granular bases are graded crushed stone bases into which a small proportion of cement, usually 4-6 per cent, has been mixed to increase dynamic stiffness and reduce susceptibility to further compaction under traffic. As its name implies, lean concrete is a weak concrete containing about 10 per cent of cement and exhibiting a compressive strength of 10-15 N/mm^2 at 7 days. In the United Kingdom, specifications for both materials are contained in the Department of Transport's General Specification for Roads.

These materials are laid without joints. Internal stresses build up within them associated with the hydration of the cement and with temperature changes. Inevitably they crack to relieve these stresses and, if the materials are not covered by a substantial thickness of upper base and surfacing, these cracks will reflect through to the surface allowing water to enter the pavement and penetrate to the foundations. Since one of the purposes of these materials is to enhance the strength of flexible pavements laid on weak foundations, these cracks are a disabling feature. For this reason, cement-bound granular and lean concrete bases are less popular than they once were.

There is one tropical environment in which this weakness can be discounted. In low lying areas of unconsolidated soil there is a need for low-cost distribution roads with surfaces often only a metre or so above flood level. In these circumstances a pavement with layers of monolithic rigid construction has advantages. In these regions, diurnal and seasonal temperature changes are often quite small, for example, in the rice bowl of Thailand and in coastal areas of Malaysia and Indonesia. The incidence and intensity of cracking induced by temperature changes are likely to be quite small and any water entering through cracks is not likely to weaken already saturated subgrades. Concrete pavements are one obvious choice in this situation but, if roads with bituminous surfacings are preferred, then the monolithic support provided by a base of cement-bound granular material or lean concrete has advantages over other forms of base.

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9 Pavement design

9.1 General principles

The elements of a flexible pavement and a rigid pavement are illustrated in Fig. 9.1.

Although the two forms of construction achieve their purpose by somewhat different means, there are common features in designing them. With both forms of construction the bearing capacity of the soil must be determined and estimates must be made of the expected traffic loading. With both, the objective of pavement design is to determine the most economical choice for the nature and thickness of the different layers of the pavement. With both, the sub-base provides a working platform on which construction plant can operate. With flexible pavements the sub-base also contributes to the strength of the road structure. This contribution is of less significance with concrete pavements.

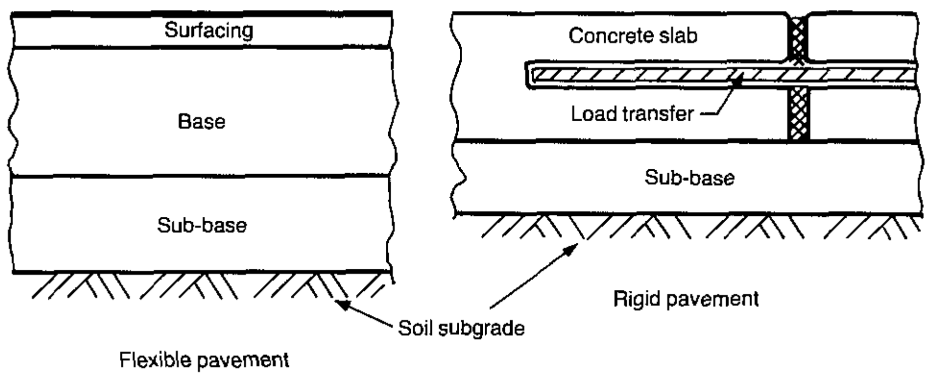


Fig.9.1 The elements of rigid and flexible pavements.

Thereafter, the methods of designing the two forms of pavement diverge. With flexible pavements it is a matter of determining which of a wide variety of possible materials are available locally to make the base and surfacing, and in what manner they should best be used. With concrete pavements the prime considerations are to determine the appropriate thickness of slab and the spacing needed for longitudinal and transverse joints to relieve the stresses developed by diurnal and seasonal temperature changes.

There are other important differences. A properly constructed concrete pavement needs very little routine maintenance thereafter but cracks develop in the slabs over the years. Ultimately the pavement has to be replaced or broken up in situ to form a base on which a new pavement is laid.

It is usual to design concrete pavements so that this condition is not reached for forty years or so. In favourable circumstances e.g. in equable climates where diurnal and seasonal temperature changes are small, they may last for much longer. The section of concrete road connecting Bangkok with the international airport, for instance, continues to give good service after over forty years, despite being built over a highly compressible soil.

Effective maintenance is a necessity with flexible pavements. The surface profile is likely to deteriorate under the compacting loads of traffic, particularly when this traffic is heavy and runs in well defined wheel tracks. This deterioration may be increased by the effects of weathering, especially in hot climates. Should the deterioration reach the stage that water enters the pavement structure through the surfacing, rapid deterioration of the upper layers of the pavement is likely to follow under the powerful disruptive forces developed by traffic stresses in the saturated material. This might be interpreted as a disabling feature of flexible construction but this is not so. In most countries over 90 per cent of the pavements are of flexible construction, and where they remain in good condition it is partly because care is taken to make sure that their surfaces are kept sealed against the entry of surface water. In many rural roads in the United Kingdom, the original pavements laid at the time of Macadam and Telford can still be found giving good service a third of a metre or so below the present surface. In India there are still lengths of the Grand Trunk Road built by the Moghuls providing foundations for present day roads with bituminous surfacings. Indeed, it seems that provided flexible pavements can be designed, built and maintained in such a way that water cannot enter the pavement structures, their life can be indefinite, requiring only correction to a deteriorating road surface profile and sufficient strengthening and widening as is necessary to accommodate increasing traffic loads. New flexible pavements are designed on the assumption that these operations will become necessary after a period of 15-20 years. Within this period it is likely that at least one surface treatment will be required to maintain impermeability and to arrest any incipient disintegration of the surface.

9.2 Soil subgrades

The load carrying capacity of a soil is probably most accurately determined by some form of plate bearing test. In the standard form of such tests, a circular plate, usually of 760 millimetres diameter, is loaded incrementally whilst the vertical displacement of the plate is measured. Plate bearing tests are used on airfields to determine the maximum wheel loading that the airfield pavement can carry safely. Such testing is tedious and there is no purpose in using plate bearing tests to evaluate the soils in their natural condition along the length of a planned road. The evaluation of the strength of a soil subgrade must take into account the state to which the soil will be compacted and the moisture conditions that will prevail under the completed road.

The California Bearing Ratio (CBR) test is most commonly used for this purpose. As originally conceived, this test was used to evaluate roadbases made from gravel or crushed stone. Specimens were immersed in water for four days before testing and a CBR value of 100 per cent indicated a satisfactory material. When the test began to be used to evaluate soil subgrades, many people continued to include the soaking procedure because of the prevailing pessimism about the possibility of preventing surface water from entering the pavement structure. There are circumstances when soil subgrades are likely to be very wet i.e. when there is a water table immediately under the pavement, but in many circumstances, particularly in hot climates, soil subgrades are quite dry and tests on soaked specimens can grossly undervalue their load bearing capacity. Nowadays the test is used in evaluating the effects of compaction and moisture content on the strength of the soil, arriving at a design CBR value by estimating the likely state of compaction and the moisture conditions that will prevail under the completed road. In planning the treatment of existing pavements, in situ measurements of the CBR of the existing subgrade can be used to give a direct measure of the strength of the subgrade provided it can be assumed that prevailing moisture conditions under the road have reached an equilibrium condition.

The effects of compaction and moisture content on the CBR values of four different soils are shown in Figs. 9.2 to 9.5. Two of these soils, namely the heavy clay and the silty clay, would be rated as rather poor subgrades. The other two, namely the well-graded sand and the gravel-sand-clay, would make excellent subgrades and, under some circumstances, would be suitable for use as sub-bases or even as bases e.g. stabilised with cement or lime. With all four soils the CBR varies considerably with variations in compacted density and moisture content. It is clearly misleading to characterise a soil with a CBR value without quoting the relevant state of compaction and moisture content. Using charts of this type for local soils, appropriate CBR design values can be derived from a knowledge of their likely state of compaction and the critical moisture conditions likely to occur under the pavement. With clayey soils it is likely that a state of compaction of 95-100 per cent of the maximum obtained in the BS Standard Compaction Test will be specified but higher levels of compaction may be specified with soils of lower plasticity (soil compaction is considered in Chapter 11). At the design stage it is appropriate to estimate CBR values at the minimum relative density likely to be achieved in the field. It then remains to estimate the critical moisture conditions that will occur in the soil immediately below the pavement.

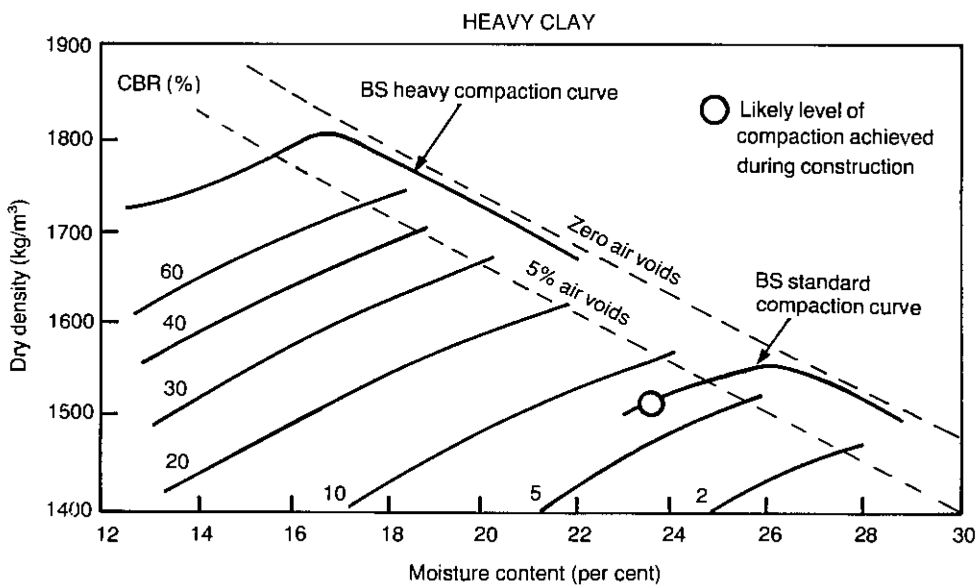


Fig.9.2 Dry density, moisture content, soil strength relationship for a heavy clay.

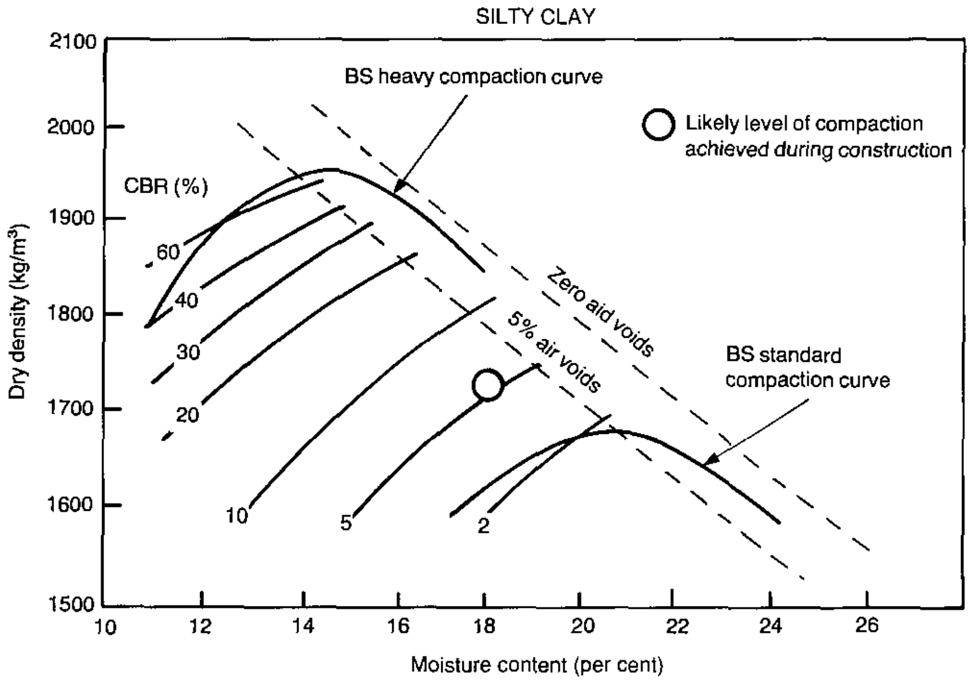


Fig.9.3 *Dry density, moisture content, soil strength relationship for a silty clay.*

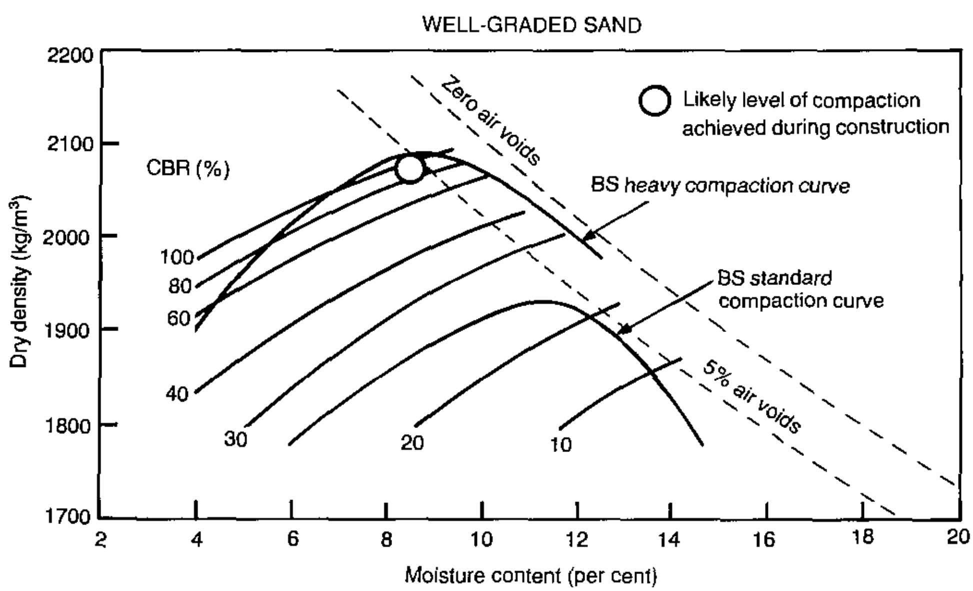


Fig.9.4 *Dry density, moisture content, soil strength relationship for a well-graded sand.*

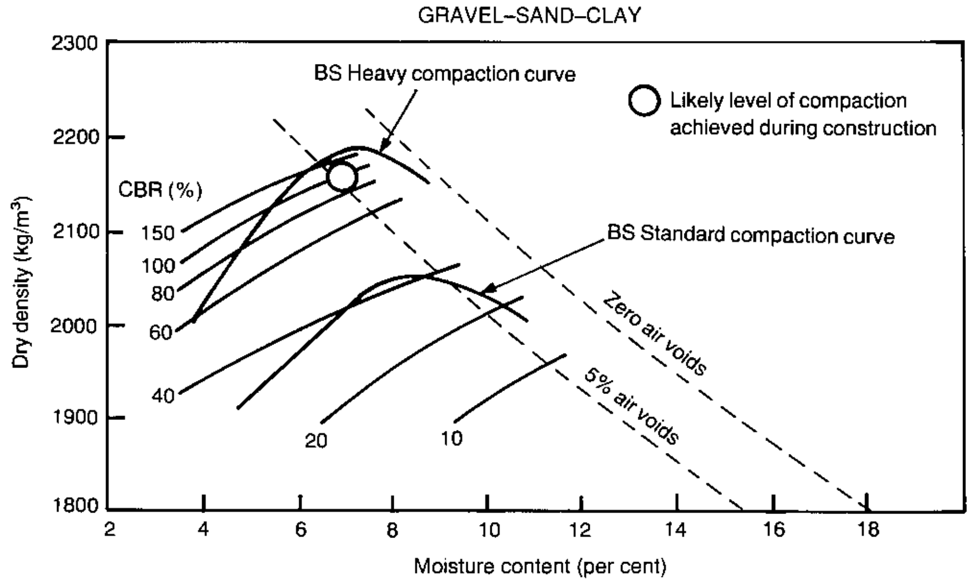


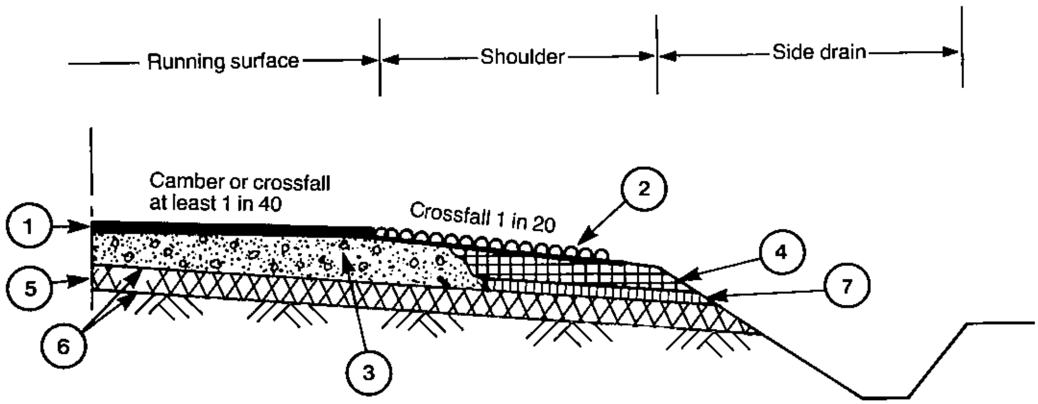
Fig.9.5 *Dry density, moisture content, soil strength relationship for a gravel-sand clay.*

9.2.1 Soil moisture

The data in Figs. 9.2 to 9.5 illustrate the condition of the soil at the time of construction. Will it change thereafter? In a dry environment the subgrade may well get stronger because of the combined effects of further compaction by traffic and a reduction in moisture content. In this case the critical CBR value to be used for design is the value at the time of construction. Subgrades under old roads are often found to be surprisingly strong. In Kenya, for instance, under pavements that had been in place for some ten years, subgrades of montmorillonitic clay were found to have in situ CBRs of 12-15 per cent. Dry densities were similar to those at the time of construction but moisture contents were somewhat lower. The design CBRs had been between 3 and 5 per cent, reflecting the strength of the soil at that time.

There are also circumstances where the moisture content of subgrades can increase, for example, if the arrangements for the drainage of surface water from the pavement structure are faulty. This was common in 'trench' construction i.e. when pavements were constructed between impermeable shoulders and with inadequate provision for carrying rainfall run-off away from the pavement into the side ditches. The pavement cross section should be designed and built to provide unobstructed paths for this run-off to pass into the side ditches. Some methods of doing this are illustrated in Fig. 9.6.

Trouble can also occur where roads intercept perched water tables which develop during wet seasons near the ground surface. Such perched water tables are likely to occur in residual soils where leached clay weathering products accumulate in a clay rich zone at a shallow depth below



1 Impervious surfacing
2 Shoulders surface dressed (giving contrasting texture to running surface)
3 Base extending under shoulder for at least 500 mm
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 in the outside wheel track)
7 Drainage layer of pervious material

Fig.9.6 *Some methods of preventing surface water run-off from penetrating road pavements.*

the ground surface. These zones occur, for instance, in the sand-veldt areas of Zimbabwe, where they are derived from the weathering of granites, and also in southern Tanzania, where they are derived from the weathering of other acid rocks. Typically, they produce zones of weakness at the ends of cuttings where the formation line of new roads intercepts the perched water table (see Fig. 9.7).

The terrain evaluation techniques described in Chapter 3 can assist local experience in identifying soil formations in which such perched water tables are likely to occur. Pavement damage can usually be avoided by making the invert of side drains at least 1.5 metres below road formation level but in extreme cases it may be necessary to install land drains diagonally across the road at the level of the invert of the side drains.

The above are matters of simple engineering, taking advantage of the fact that gravity will always remove surface water from the vicinity of road pavements, given available flow paths. There are two other situations which must be considered. One is in low lying ground where the natural water table lies close to the surface. The other occurs in monsoon climates and in climates of persistent high humidity where, for at least part of the annual climatic cycle, large amounts of ground water can occur in the soil above the level of any water table that is present.

Water is continually moving through the soil mantle in all parts of the world (except in areas of permafrost where it is fixed in place as ice). In rainy weather it moves downwards to join the water

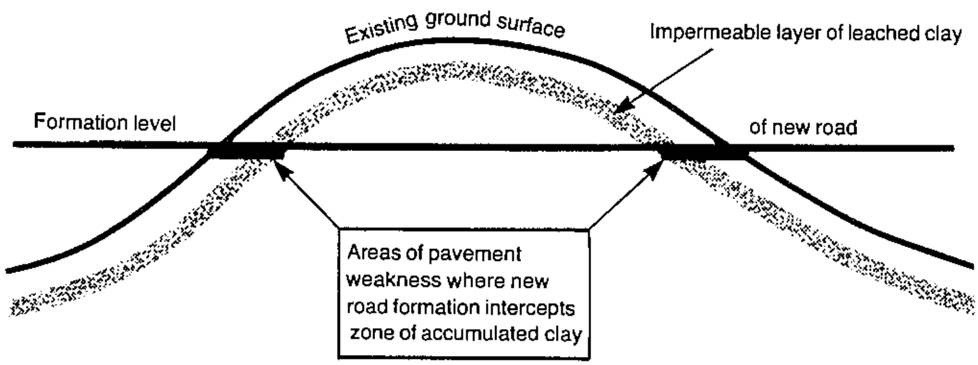


Fig.9.7 Pavement failures can be caused by perched water tables in residual soils.

table, if one exists. In dry weather the predominant movement is upwards, with water evaporating directly from bare soil and being lost by transpiration from vegetation. Some of this upward movement is as water vapour. In desert areas, for instance, underground water, fed by aquifers from distant mountains, is not unusual. Turning over a flat stone on the surface of a desert in the early morning reveals a damp underside which is evidence of this continual movement of moisture upwards, condensing under the stone in the cool of the night.

Capillary action draws water upwards in soil for a small distance above a water table. With clayey soils, other more powerful forces are active. These are the suction forces which generate the cohesion in clayey soils at moisture contents below the liquid limit (see Chapter 2). Where roads are built over ground with water tables less than 7 metres below the surface, it is likely that the presence of the water table will be the dominant influence on moisture conditions in the soil subgrade. The moisture content will move towards an equilibrium value which may be wetter than at the time of construction and this must be taken into account in determining the design value for the CBR of the soil. This aspect of the behaviour of clayey soils was put onto a quantitative basis for road engineers by Croney *et al.* (1958).

Fig. 9.8 illustrates the influence of a water table at a depth of 2 metres below a sealed surface. Immediately below this surface the non-plastic sandy gravel is quite dry whilst the clayey soils are wetter than immediately above the water table because of the increased overburden pressure with depth. It is this mechanism which determines the critical moisture conditions and the strength of soil subgrades over much of the United Kingdom and in other regions where there are permanent water tables less than 7 metres below the surface of roads.

In many tropical and sub-tropical areas, water tables are at considerably greater depth below the surface. A study of moisture conditions under airfields in East Africa and the Middle East, reported by Russam and Croney (1960), together with information on moisture conditions under roads in many tropical countries, has produced a method of determining critical moisture conditions in soil subgrades where there is no shallow water table. As a generalisation, and provided that the roads are properly engineered, in the absence of a shallow water table, soil

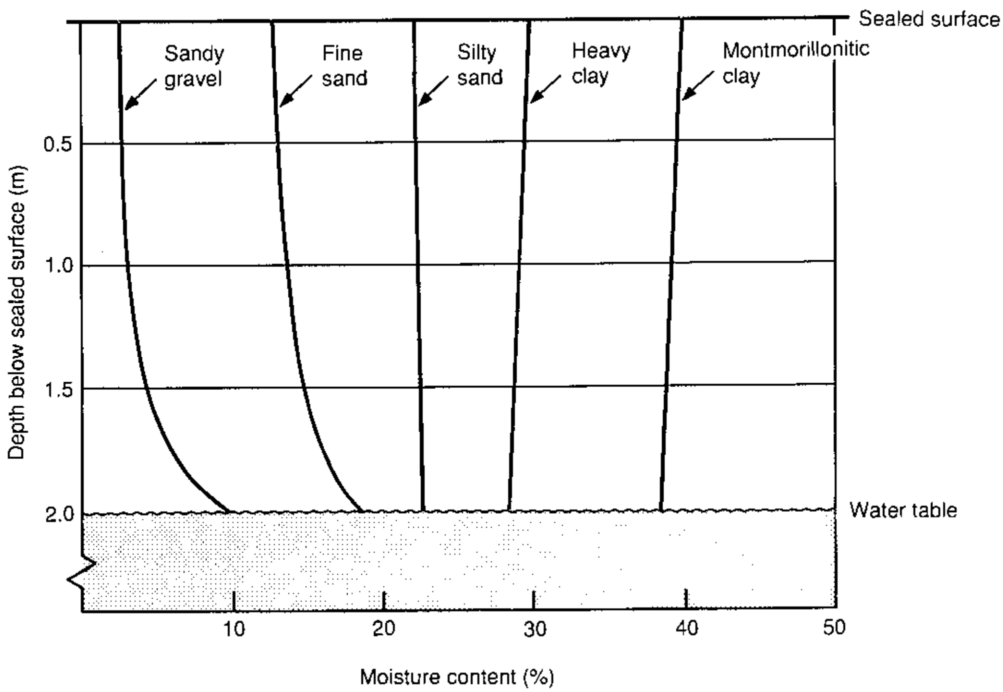


Fig.9.8 *Equilibrium moisture contents with water table 2 metres below sealed surface.*

subgrades are never likely to get wetter or weaker than they are at the time of construction. In many circumstances they are likely to get drier and stronger.

In Road Note 31, the guide to the structural design of bitumen-surfaced roads in tropical countries issued by the Transport and Road Research Laboratory, environments are classified into three categories in which the influences controlling moisture conditions in soil under roads are markedly different. These three categories, illustrated in Fig. 9.9, are as follows:

Category 1. Water table within 7 metres of the level of the finished road surface.

Measurements of soil suction, as described by Croney and Croney (1991), can be used to estimate equilibrium moisture contents of the soil at different heights above the water table. Unfortunately, in normal design testing, neither the apparatus nor the necessary skilled staff are likely to be available. Quicker and simpler methods must be used. On sites where shallow water tables are suspected, boreholes should be made at critical points during the survey and the water levels observed in them, preferably over a complete seasonal cycle. If there are existing roads or other covered areas in the vicinity, measurements can be made of prevailing moisture conditions under them. Such covered areas should be at least 3 metres wide and should be at least two years old. At each particular height above the water table, the ratio of soil moisture content to plastic limit is constant for soils of the

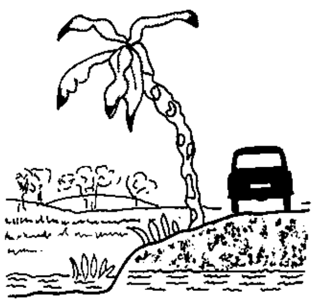
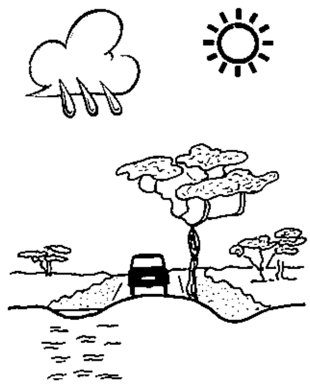
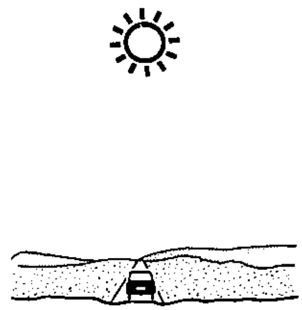
CATEGORY 1	CATEGORY 2	CATEGORY 3
WATER TABLE WITHIN 10m OF ROAD SURFACE	DEEP WATER TABLE MONSOON CLIMATE Annual rainfall 250mm or more	DEEP WATER TABLE ARID CLIMATE Annual rainfall less than 250mm
		
Subgrade moisture and strength determined by depth to water table	Subgrade moisture content not likely to exceed optimum in BS Standard compaction test	Subgrade moisture conditions as in uncovered soil at the same depth

Fig.9.9 *Climatic categories determining soil subgrade moisture conditions.*

same pedological type. Measurements made under an existing road can therefore be translated to indicate the equilibrium moisture contents for design purposes of similar soils. Table 9.1 gives the estimated minimum CBR values for six subgrade soils with water tables up to a depth of 7 metres below formation level and with the soils compacted to at least 95 per cent of the maximum density obtained in the British Standard (Light) Compaction Test (2.5kg rammer method). This Table may also be used to make the necessary judgements on appropriate design CBR values but it is not as precise as the other methods described above.

Category 2. Deep water table and monsoon climate.

In such environments most of the rain water is removed from the vicinity of the road by surface run-off but some remains to move downwards through the soil profile. This ground water will have little effect on moisture conditions under the road and, provided the surface water drainage facilities are kept in good working order, it may be assumed for design purposes that the moisture content of the soil subgrade will not exceed the optimum moisture content in the British Standard (Light) Compaction Test (2.5kg rammer method).

TABLE 9.1
ESTIMATED *MINIMUM* DESIGN SUBGRADE CBR VALUES UNDER PAVED ROADS. (FOR SUB-GRADES COMPACTED TO 95% OF THE MAXIMUM DRY DENSITY OBTAINED IN THE BS STANDARD COMPACTION TEST).

Depth of water-table* from formation level	Minimum CBR (per cent)					
	Non-plastic sand	*Sandy clay PI = 10*	*Sandy clay PI = 30*	*Silty clay PI = 30*	*Heavy clay PI ≥ 40*	*Silt*
0.6m	8	5	4	3	2	1
1.0m	25	6	5	4	3	2
1.5m	25	8	6	5	3	See note (d)
2.0m	25	8	7	5	3	"
2.5m	25	8	8	6	4	"
3.0m	25	25	8	7	4	"
3.5m	25	25	8	8	4	"
5.0m	25	25	8	8	5	"
7.0m or more	25	25	8	8	7	"

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

NOTES:

a) Since the values given in Table 9.1 are estimated minimum CBR values, wherever possible the CBR should be measured by laboratory testing at the appropriate moisture content.

b) 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 sub-grade CBR for these soils will be obtained by using the values quoted above for sandy clays.

c) This table cannot be used for soils containing appreciable amounts of mica or organic matter.

d) Laboratory CBR tests are required for pure silt subgrades with water-tables deeper that 1.0m.

In Categories 1 and 2 there are circumstances where it may be appropriate to base designs on CBR values measured on saturated samples. In Category 1, for instance, roads often need to be built at a low level through areas of wet padi cultivation. In Category 2 there may be circumstances where there are doubts about the practicality of preventing surface water from penetrating under the road, for example, in climates of persistent high humidity or where there are grounds for believing that it will not be possible to provide a pavement through which surface water cannot penetrate.

Category 3. Deep water table. Arid climate

Where annual rainfall does not exceed 250 millimetres per year, moisture conditions under a sealed surface are likely to be the same as in exposed soil at the same depth and it may be assumed that subgrade strengths are always likely to be as high or higher than those obtained from CBR tests on the soils at the densities and moisture contents at the time of construction.

Moisture movement upwards in the vapour phase undoubtedly occurs under these conditions but, with the sandy non-plastic soils common in these areas, it seems likely that moisture condensing under sealed surfaces drains away without prejudicing the strength of the road structure. Where clays, particularly expansive clays, are exposed on the surface, a different situation may occur. Ruckman (1980) has reported that in building roads over montmorillonitic clays in Colorado, it was found necessary to wet the soil subgrades during construction in order to induce the swelling that otherwise would have occurred under the sealed surfaces because the natural evaporation of water, moving up through the soil as water vapour, had been prevented.

Engineers working in a particular area acquire a familiarity with local conditions which they are able to use in making the judgements that are necessary in defining design values for the CBR of soil subgrades. But in many developing countries, government engineers are thin on the ground and are often frequently transferred from one area to another, thus preventing them from building up this local experience. Consulting engineers competing for project design and contract supervision may have little or no experience of the areas in which they come to work. The land-form classification systems described in Chapter 2, provide a means of storing accumulated experience on prevailing moisture conditions under roads and on the influence of these moisture conditions and compaction on design CBR values for local soils.

9.3 Road traffic

Road traffic is characterised in different ways according to the use intended for the data. Traffic engineers and planners are concerned with the safe and uninterrupted flow of traffic, and they are interested in the space that different types of vehicle occupy on the road. The bridge engineer is concerned with the maximum live load on his structures and his interest is in the size and overall weight of vehicles. The pavement engineer is concerned with the magnitude of the individual loads that will be applied to the pavement and his interest is predominantly in the range of wheel loading on the vehicles that will use the road. All are involved in predicting likely rates of traffic growth so that the facilities they provide are adequate to accommodate this growth.

9.3.1 Road capacity

In industrialised countries the traffic on roads consists almost entirely of motor vehicles. Indeed, in the USA, to be seen walking along a public road outside urban areas is to risk being apprehended on suspicion of vagrancy. In some countries, the Netherlands for instance, many people travel by bicycle and special tracks are often provided for them. In developing countries, particularly in Africa and Asia, the mix of traffic is much more varied. In Asia there are animal-drawn vehicles and, sometimes, pack animals using the roads. In Africa this traffic is generally absent, with some notable exceptions such as the animal-drawn vehicles in Egypt, camel trains in the Sahel, and horses in Lesotho. Everywhere near centres of human habitation, there are people on foot and on bicycles using the roads.

This heterogeneous mixture of traffic has two important effects. Firstly, it is a main reason for the grievous toll of road accidents in developing countries. Secondly, the relationships between pavement width and traffic capacity developed in the industrialised countries have to be modified for use on all-purpose roads in developing countries. Information on geometric road standards is outside the scope of this book. For those wishing to pursue this subject further, the references to Kosasih *et al* (1988) and Overseas Road Note No 6 (Transport and Road Research Laboratory (1988)) will be of interest.

9.3.2 Vehicle loading

In most developing countries, trucks carry loads much in excess of their rated capacity. Indeed, in many developing countries there is a thriving industry at ports of entry occupied in fitting wider and higher bodies to imported trucks. So equipped, a truck can carry over twice its rated payload, thus producing a considerable reduction in perceived haulage costs. In some countries it is not uncommon to find two-axled trucks with all-up weights of up to 30 tonnes and with tyre pressures much in excess of normal values. The wheel loads and contact pressures that result are of the same order as those applied by the compaction equipment during construction. Thus, when the vehicles run in well defined wheel tracks, it is not surprising that newly constructed flexible pavements, particularly those with unbound bases, frequently show signs of distress shortly after they are opened to traffic. On flexible pavements with bases bound with bitumen or cement, and on concrete roads, the immediate effects of this overloading are less obvious but their service lives are considerably reduced below normal expectation. Rolt (1981) has examined the implications of vehicle overloading on overall transport costs and it is evident that vehicle overloading is seriously handicapping the improvement of the road networks in many developing countries. An indication of the extent of this overloading can be seen from the data in Table 9.2.

TABLE 9.2
EXAMPLES OF AXLE LOAD SPECTRA MEASURED ON MAIN ROADS

Country	Percentage of axle loads above				
	8 tonnes	*10 tonnes*	*12 tonnes*	*14 tonnes*	*16 tonnes*
Ethiopia	45*	34	24	15	
Jordan	77	58	45*	28	14
Kenya	67*	52	30	4	
Nigeria	-	30*	10#		4
Qatar	43	36	27	14	13
Turkey	28*	17	7	4	
W Malaysia	12*	4	2		
UK	7.5	4*	3	2	

* Legal limit of axle load (tonnes)
\# Percentage at 13 tonnes.

All industrialised countries have vehicle construction and use regulations prescribing limits on the size and weight of commercial vehicles and enforcement is quite strict. In Western Europe these regulations are being unified under the auspices of the European Community. For the two and three-axle trucks that predominate in most developing countries, the proposed E.C. limits are summarised in Table 9.3.

TABLE 9.3
PROPOSED E.C. LIMITS ON SIZE AND WEIGHT OF TWO AND THREE-AXLED VEHICLES

Vehicle	*Length (m)*	*Width (m)*	*Gross weight (tonnes)*	*Axle load (tonnes)*
Two-axle rigid	12.0	2.5	18	11.5
Three-axle rigid	12.0	2.5	25	11.5
Three-axle articulated	16.5	2.5	(1)	(1)

(1) No specified limit. At the discretion of member states.

There is now a wide range of multi-axle vehicles available and, with the advent of container-traffic, more of these are coming into greater use on the roads in developing countries. The proposed E.C. loading limits for them are given in Table 9.4.

In the USA the regulations concerning the size and axle loading of commercial vehicles vary considerably from State to State. Although limits on gross vehicle weights are generally higher than in Europe, maximum axle loadings are generally lower. This perhaps reflects the vigour with which State Highway Departments have striven to protect their road pavements. In both Europe and North America the regulations on vehicle dimensions and loading are quite strictly enforced. Some developing countries have similar regulations but, with very few exceptions, it has proved quite impossible to enforce them. The spectra of axle loading in most countries are far heavier than on roads in Europe and North America. It seems unlikely that the level of enforcement will

TABLE 9.4.
PROPOSED E.C. LOADING LIMITS FOR LARGE COMMERCIAL VEHICLES

Vehicle Type	*Axle load (tonnes)*			Gross vehicle weight (tonnes)	Pay load (tonnes)
	Tractor		Trailer		
	Steer	*Drive*	All axles		
Two-axle tractors with two-axle trailers	6	11	18	35.0	24.0
Two-axle tractors with three-axle trailers	6	11	23	40.0	28.0
Three-axle tractors with two-axle trailers					
Single drive	6	11 and 7	18	42.0	29.6
Double drive	6	18 combined	18	42.0	29.6
Three-axle tractors with three-axle trailers					
Single drive	6	11 and 7	20	44.0	31.2
Double drive	6	18 combined	20	44.0	31.2

improve very quickly and road pavements must be designed so that they can carry vehicle loads that are much heavier than those operating in industrialised countries.

9.3.3 Effects of vehicle loading on pavement design

The most widely used relationship between vehicle loading and pavement performance was derived from the AASHO Road Test (Highway Research Board (1962) and W J Liddle (1962)). This test involved running vehicles of different loading characteristics, for a period of up to two years, over test tracks containing lengths of flexible and concrete pavement of different formulations and observing and measuring the condition of the pavements as they deteriorated under the traffic loading. The test was carried out over the period 1958-60 at Ottawa, Illinois, where the subgrade soil is a wind-blown loess and where weather conditions are typical of the Northern USA i.e. a continental climate with hot summers and cold winters. The data were subjected to a complex statistical analysis and amongst the results which emerged was a generalised conclusion that the *relative* damage to both flexible and rigid pavements varied approximately as the fourth power of the applied wheel load. It is this relationship that provides the basis for assessing the effects of vehicle loading in most current methods of pavement design. The relationship was codified by converting the estimated spectra of axle loadings into an equivalent number of repetitions of a standard axle load of 18,000 lb (8160 kg). The factors for this conversion to Equivalent Standard Axles (ESA) are shown in Table 9.5 and an example of its use is shown in Fig. 9.10. The relationship is often known simply as the fourth power law. It

TABLE 9.5
FACTORS FOR CONVERSION OF AXLE LOAD TO NUMBERS OF EQUIVALENT STANDARD AXLES

Wheel load (kg)	*Axle load (kg)*	*Equivalence factor*
1500	3000	0.01
2000	4000	0.04
2500	5000	0.11
3000	6000	0.25
3500	7000	0.50
4000	8000	0.91
4500	9000	1.55
5000	10000	2.50
5500	11000	3.83
6000	12000	5.67
6500	13000	8.13
7000	14000	11.3
7500	15000	15.5
8000	16000	20.7
8500	17000	27.2
9000	18000	35.2
9500	19000	44.9
10000	20000	56.5

Equivalence factor = $(\text{Axle load in tonnes} / 8.16)^{4.5}$

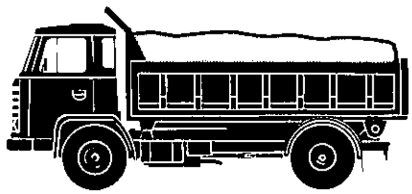
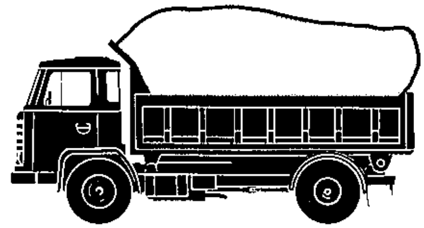
PROPERLY LADEN VEHICLE	OVERLADEN VEHICLE
	
ALL UP WEIGHT 16 tonnes	ALL UP WEIGHT 28 tonnes
AXLE LOADS 6t 10t	AXLE LOADS 9t 19t
EQUIVALENT STANDARD AXLES 0.25 2.50 Total ESAs = pavement damaging effect 2.75	EQUIVALENT STANDARD AXLES 1.55 44.9 Total ESAs = pavement damaging effect 46.4
PAY LOAD 10 tonnes	PAY LOAD 22 tonnes

Fig.9.10 *Effects of vehicle overloading on payload and pavement damage.*
(In this example the overladen vehicle exerts a damaging effect on the road pavement which is more than sixteen times greater than that produced by the properly laden vehicle.)

has been adopted almost universally and has become invested with something of the sanctity of a natural law. This it certainly is not. The heaviest axle load used in the Road Test was 10.7 tonnes and the equivalence factors for higher axle loads were derived by extrapolation.

Furthermore there is no *a priori* reason to believe that the fourth power relationship should necessarily apply to roads experiencing different climatic conditions from those that pertained at the Road Test, to roads with different structures and to roads which deteriorate through different mechanisms. A review of the effects of axle loads on pavement damage is included in Paterson (1987).

In most developing countries the magnitude of vehicle overloading is such that specific precautions need to be adopted to minimise its deleterious effects. One defence is to make sure that both new and rehabilitated pavements are designed using a realistic assessment of the

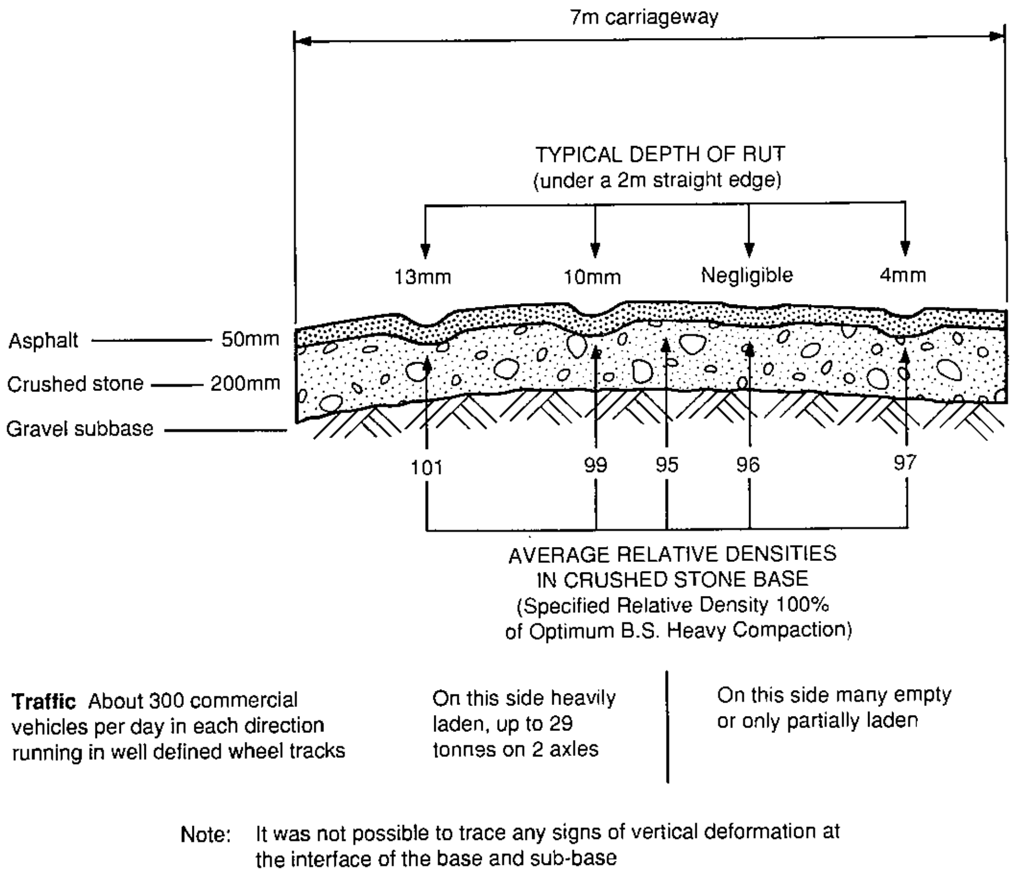


Fig.9.11 Investigations of rutting on a newly constructed length of the Islamabad-Peshawar road, Pakistan.

expected traffic loading. On flexible pavements this precaution may not be enough when the traffic contains a high proportion of grossly overloaded vehicles running in well defined wheel tracks. Such traffic is likely to produce rapid differential compaction in the upper layers of the pavement. An example is illustrated in Fig. 9.11. On this road carrying a concentration of very heavily loaded vehicles, severe rutting became evident within a year of opening, in some places rupturing the asphalt surfacing. Despite a fairly rigorous specification for the compaction of the dry stone base, the very heavy wheel loads had produced further compaction of the base in the wheel tracks. On a similar, newly constructed road within a nearby urban area the heavily laden vehicles were mixed with pedestrians, cyclists, carts and other vehicles. There were no well defined wheel tracks and the pavement remains in good condition. Here, we have, in a somewhat exaggerated form, one of the typical modes of deterioration of flexible pavements. Such pavements are normally designed in the expectation that such deformation in the wheel tracks will develop over the design period of 15-20 years and will be one of the reasons for resurfacing and possible strengthening. The case described above is no doubt extreme, but there have been many cases of roads with similar construction carrying overladen vehicles in well defined wheel tracks that have reached a similar condition after three or four years instead of the expected design life of 15-20 years.

What are the remedies? One remedy would be to use the same designs as for the very heavily constructed pavements employed in ports and other industrial areas to carry heavily laden gantry cranes and fork-lift trucks. Such pavements are much stronger and correspondingly more expensive than those normally used on roads and this solution would be a great handicap to road building in countries afflicted with overladen vehicles.

A more practical solution is to make sure that during construction the upper layers of pavements are extremely well compacted using all the techniques available to do this and to treat any distress caused by the overladen vehicles as soon as it occurs. The form of this distress and its treatment will vary according to the climatic environment, the nature of the pavement construction and the degree of overloading.

With unbound bases the primary effect will be in further compaction of the pavement layers. This further compaction is beneficial and can add to the strength of the pavement. But where the vehicles run in well defined wheel tracks, differential compaction will occur and shear cracks may develop in the wheel tracks, particularly if there are any shortcomings in the compaction of the base at the time of construction. Furthermore, rapid weathering of the bitumen in the asphalt surfacing is likely to promote cracking of the surface under both the permanent rutting in the wheel tracks and the transient deformations (deflections) under the heavy traffic loads. In a dry environment, such cracks are not likely to be the cause of structural weakness, and repair is necessary only when the profile of the running surface deteriorates to a level that it interferes with the free flow of traffic.

In wet and monsoon climates, the situation is different. As cracks develop, water can enter the pavement through them. During prolonged wet weather, this water accumulates in the upper layers of the pavement and powerful hydrodynamic stresses are developed under the passage of heavy vehicles which cause disruption of the surfacing and upper layers of the base. Again, this sequence is more rapid and extensive when there have been any shortcomings in compaction at the time of construction. It is vital that any cracks that appear are sealed to prevent the penetration of water. This can be done with a surface dressing which not only seals the cracks but also reduces further weathering in the underlying asphalt. In addition, the surface dressing is able to tolerate the transient deformations of the pavement under load and to carry the pavement successfully through the period in which the heavily loaded vehicles are producing further compaction of the upper layers of the pavement. There is evidence that if damage can be prevented during the first five years of the life of a pavement, the pavement will thereafter gain rather than diminish in dynamic stiffness.

In wet environments, particularly on more heavily trafficked roads, there are advantages in using bases bound with bitumen or cement to minimise the differential compaction under overloaded vehicles. Nevertheless, the structural life of the roads is likely to be impaired by the overloading which may not be taken fully into account by the fourth power relationship. For low cost roads in humid climates, particularly those built over unconsolidated soils, the relationship may under-rate the damage caused by overloaded vehicles. Conversely, in hot dry climates it is likely that the damaging effects of overloaded vehicles is less than indicated by the fourth power relationship. But, within the limitations outlined above, the fourth power relationship gives at least an approximate indication of the damaging effect of heavy vehicles for expressing traffic loading for pavement design purposes.

9.3.4 Estimation of traffic loading

For pavement design purposes the loads applied by private cars and other light vehicles are of no consequence. Only heavy vehicles that have axle loads of 3 tonnes or more need be considered i.e. the larger commercial and public service vehicles. However, on most roads outside larger towns and cities, public service vehicles form only a very small proportion of the traffic, therefore it is normal practice to calculate traffic loading for pavement design from estimates of the number of heavy commercial vehicles using the most heavily trafficked lane over the design period. These estimates are converted to numbers of Equivalent Standard Axles. The essential components of these calculations are:

1) Measuring (for existing roads) or estimating (for new roads) the number of heavy commercial vehicles per day using the road at the present time.
2) Predicting the likely traffic growth over the design period.
3) From 1) and 2) deriving the total number of heavy commercial vehicles using the most heavily trafficked lane over the design period.
4) Converting this number of heavy commercial vehicles into the number of Equivalent Standard Axles.

For a heavily trafficked motorway the design loading so derived may well exceed 50 million ESA. At the other extreme, a lightly trafficked rural road will have a design loading of 0.5 million or less.

Some imprecision is inevitable in such estimates. Even in industrialised countries, where a great deal of data and experience are available on the characteristics of road traffic, there are uncertainties about future traffic flows. The effect of this particular uncertainty is not large and is likely to be lost amongst the other uncertainties affecting the long term performance of road pavements. On the other hand, incorrect estimates of vehicle loading can have a much more serious effect on the behaviour of road pavements. The experience on the Nairobi-Mombasa road, mentioned in Chapter 1, provides a graphic example. Much of the road had been rebuilt in the 1960's on the then prevailing design assumption that it was numbers of commercial vehicles that were critical rather than the loads that they applied to the road pavements. Some sections had roadbases of crushed stone and others of natural gravel. After four years of traffic, all sections were showing evident signs of distress and a major programme of pavement rehabilitation was necessary. Investigation revealed the extent to which the vehicles using the road were overloaded above their rated capacities. With the traffic loading expressed in cumulative Equivalent Standard Axles, a pavement life of four years was indicated rather than the 15 years expected at the time of the original design. It is experiences such as this that have led to the present-day insistence that road pavements must be designed on a realistic assessment of the spectra of axle loading that the pavements will carry.

The spectrum of traffic loading is likely to vary on different classes of road, with heavier vehicles predominating on the busier main roads. Vehicle damaging factors used in the United Kingdom are shown in Table 9.6. The change between 1970 and 1983 derives from the increase in the numbers of multi-axled vehicles and not from any change in axle loading.

Pavement design engineers should be able to go to national highway authorities for information on the axle loading spectra on different classes of road and it is important that highway authorities set out to obtain this information in a systematic way. Installations are now available that can be built into the wheel tracks to provide a continuous record of the wheel loads of passing vehicles. Some are equipped to take photographs of the rear of vehicles whose axle loads exceed particular limits. In developing countries it is more likely that information will be gathered by mobile teams equipped with portable weighbridges. Some cunning is needed both in siting such surveys and in their execution because the news is soon broadcast that a survey is in progress and drivers of heavy vehicles will seek out alternative routes or even postpone travelling until the survey is over. Information on the apparatus and conduct of such surveys is given in the TRRL's Road Note 40.

TABLE 9.6
VEHICLE DAMAGING FACTORS RECOMMENDED FOR PAVEMENT DESIGN IN THE UNITED KINGDOM

Category of road (commercial vehicles per day per traffic lane)	Equivalent Standard Axles per commercial vehicle *1970*	*1983*
> 2000	1.08	2.9
1000 - 2000	1.08	2.25
250 - 1000	0.72	1.25
< 250	0.45	0.75

9.4 The design of flexible pavements

It has always been an ambition to design road pavements by means of rigorous methods of structural analysis similar to those used in the design of bridges and other civil engineering structures. Some of the protagonists of this approach tend to advocate it on the grounds that it is the only truly scientific approach, implying that any other approach savours of an empiricism which they scorn. With concrete pavements it has been possible to develop analytical methods of design which are of universal application, but with flexible pavements, modes of pavement deterioration are complex and dependent on the local environment and on effective maintenance. So far, at least, it has not proved possible to produce an analytical method of universal application that takes these variables adequately into account.

This does not mean that the analytical approach has so far been unproductive. On the contrary, it has proved to be very useful in focusing research on the three properties that have a dominant influence on the performance of flexible pavements under traffic. These are:

(a) The stiffness i.e. the modulus of elasticity, of the upper layers of the pavement. It is this which determines their load spreading properties.
(b) The strength of the upper layers of pavement i.e. their ability to accommodate the strains imposed by traffic loads without rupture.
(c) The resistance of all pavement layers to further compaction under traffic loads.

The complex visco-elasto-plastic nature of the materials in the different pavement layers make it necessary to study these three properties under dynamic loading. The stiffness, strength and resistance to deformation of bituminous bases and surfacings can be investigated in the laboratory using sophisticated equipment to evaluate laboratory prepared samples and samples cut from the road and sawn to shape. There has always been the ambition to undertake accelerated pavement testing and many countries have employed circular test tracks with rolling loads tethered to a revolving arm. But these systems have two disadvantages. One lies in the unrepresentative radial forces generated by the rotation. The other is that modern traffic on busy roads produces repetitions of loading far more intense than can be simulated on a circular track.

These difficulties were first overcome in South Africa by the development of a linear heavy vehicle simulator which is mounted in a vehicle so that it can be operated both on public roads and on test sections built in circumstances where temperature and moisture conditions can be carefully regulated. This machine, initially described by Van Vuuren (1972), operates with a rolling wheel load that can be adjusted between 2000 and 8000 kilograms with a frequency of 800 repetitions per hour.

This accelerated trafficking provides a vital link between the laboratory tests on materials and their performance in road pavements. At the Transport and Road Research Laboratory in the UK, a linear trafficking test facility has been available since 1985. This facility, operating in a test pit 25 metres long and 10 metres wide, can accommodate up to ten test pavements instrumented to measure stresses and deflections under rolling loads up to 10 tonnes at speeds up to 20 km/hr. This testing facility is shown in Fig. 9.12. Such machines, operating continuously for a few weeks, can simulate the effects of the passage of one million Equivalent Standard Axles.

An example of the complexity of this subject can be illustrated by considering the dynamic modulus of asphalt mixtures. Fig. 9.13 shows the range of moduli assigned to asphalt concrete surfacings in the current AASHTO 'Guide for the design of pavement structures' (1986). These moduli apply at a temperature of 68°F (20°C) but the dynamic modulus of asphalt mixtures is much affected by temperature, as shown in Table 9.7.

TABLE 9.7
ASPHALT MODULUS VALUES
(SOURCE: OVERSEAS UNIT, TRL)

Temperature °C	Modulus (GPa)			
	High speed (1)		*Low speed (2)*	
	New asphalt	Old asphalt	New asphalt	Old asphalt
15	8.0	12	4	6.5
20	7.0	10	3	5.5
25	5.5	8.0	1.8	4.0
30	4.0	6.0	1.1	3.0
35	2.5	4.5	0.6	2.0
40	1.7	3.5	0.3	1.3

Notes: 1. Fast free flowing traffic
2. Creep speed (eg deflection beam testing)

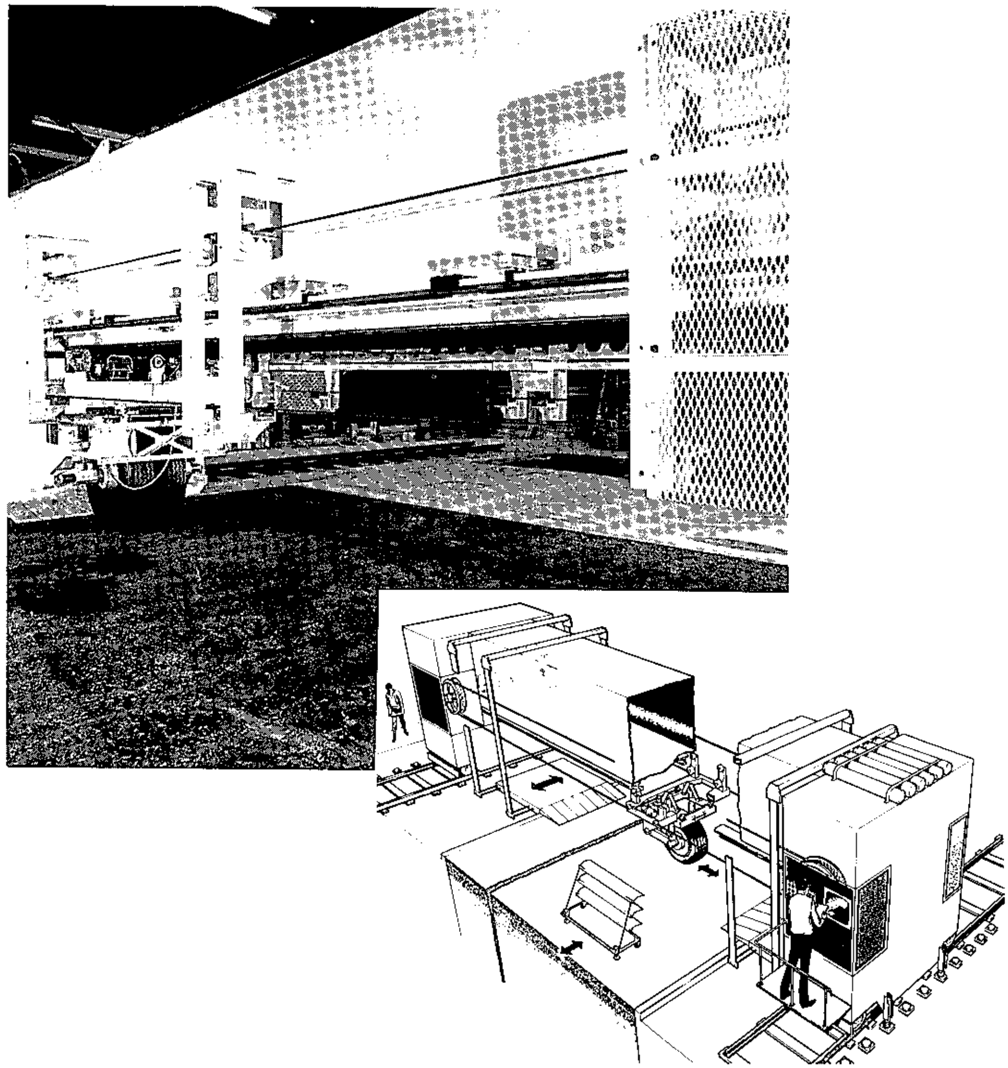


Fig.9.12 The British Pavement Test Facility (PTF) - a linear accelerated loading machine.

The Table also shows the effect of the age hardening of the bitumen on the dynamic modulus under tropical conditions. Pavement temperature is clearly important in evaluating the dynamic modulus and the associated load spreading properties. Amongst the various analytical design methods, the Shell method appears to be the only one which takes this factor into consideration at the present time (Shell International Petroleum Co. (1978)).

Current methods of designing flexible pavements rely increasingly on the accumulated knowledge of the dynamic modulus, strength, and resistance to deformation of the materials used in the different layers of the pavement. These methods vary considerably in the extent to which this knowledge is explicit in the text of the design method as published. At one time, for instance, in the French speaking countries of West Africa, pavement design was considerably simplified by

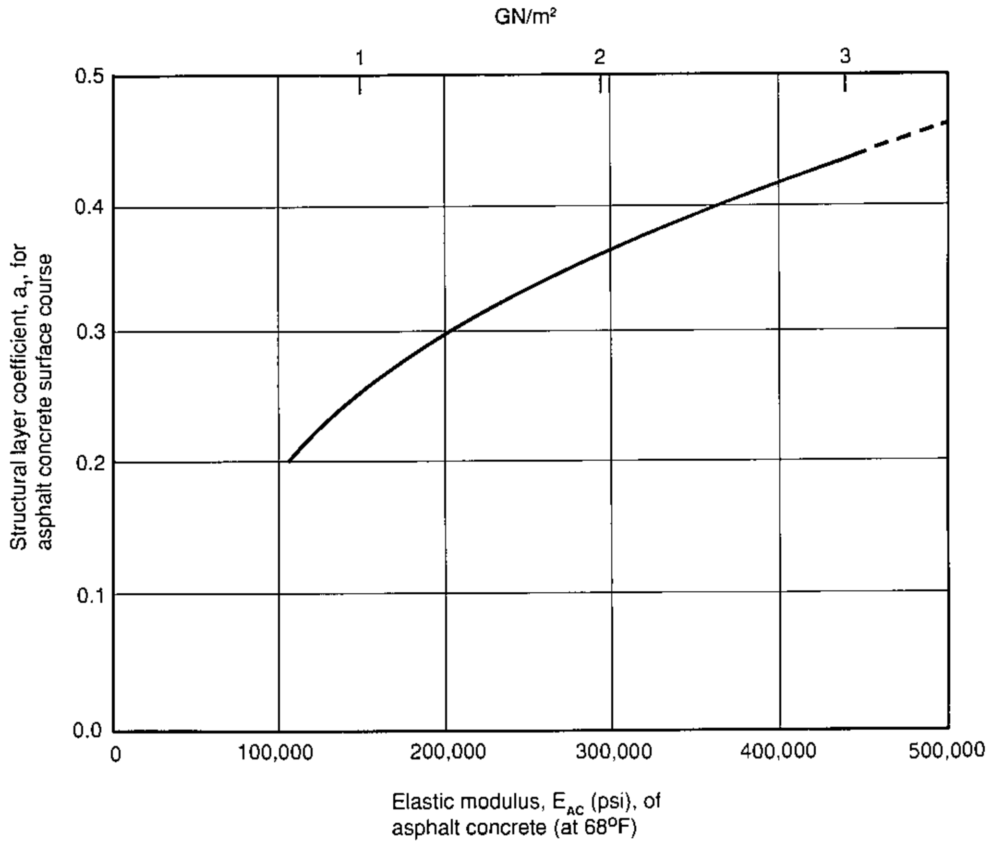


Fig.9.13 *Chart for estimating structural layer coefficients of dense-graded asphalt concrete based on resilient (elastic) modulus.*

restricting choice to six forms of pavement, the choice being made according to traffic intensity and estimated soil strength. An elaboration of this approach is used in several design methods including the new edition of TRRL's 'Guide to the structural design of bitumen-surfaced roads in tropical countries'. This contains a structure catalogue which defines the appropriate thicknesses of pavement layers as affected by the design traffic loading and the design CBR of the soil subgrade. Fig. 9.14 shows the ranges of traffic loading and soil CBR covered in the TRRL guide and also the symbols used to indicate the different forms of surfacing, base and sub-base. As an illustration of the use of the catalogue, Fig. 9.15 shows how traffic loading and soil CBR affect the thickness of layers on pavements made with crushed stone bases surfaced with a double surface dressing.

The NAASRA design method (1987) is more explicit in dealing with the variables affecting the performance of flexible pavements. Although the fourth power relationship derived from the AASHO Road Test is normally used in calculating Equivalent Standard Axles, there is latitude

ROAD NOTE 31 STRUCTURE CATALOGUE: KEY

Traffic categories
(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 categories
(CBR %)

S1 = 2 – 3
S2 = 3 – 5
S3 = 5 – 8
S4 = 8 – 15
S5 = 15–30
S6 = >30

Material Definitions

SD	Double surface dressing
AG	Flexible gap-graded bituminous surface
AC	Bituminous surface (Usually a wearing course, WC, and a base course, BC)
BB	Bituminous roadbase
GB1–5	Granular base
GS	Granular subbase
GC	Granular capping layer or selected subgrade fill
CB1	Cemented base 1
CB2	Cemented base 2
CS	Cement stabilised subbase

Fig.9.14 Key to Road Note 31 catalogue of road structures.

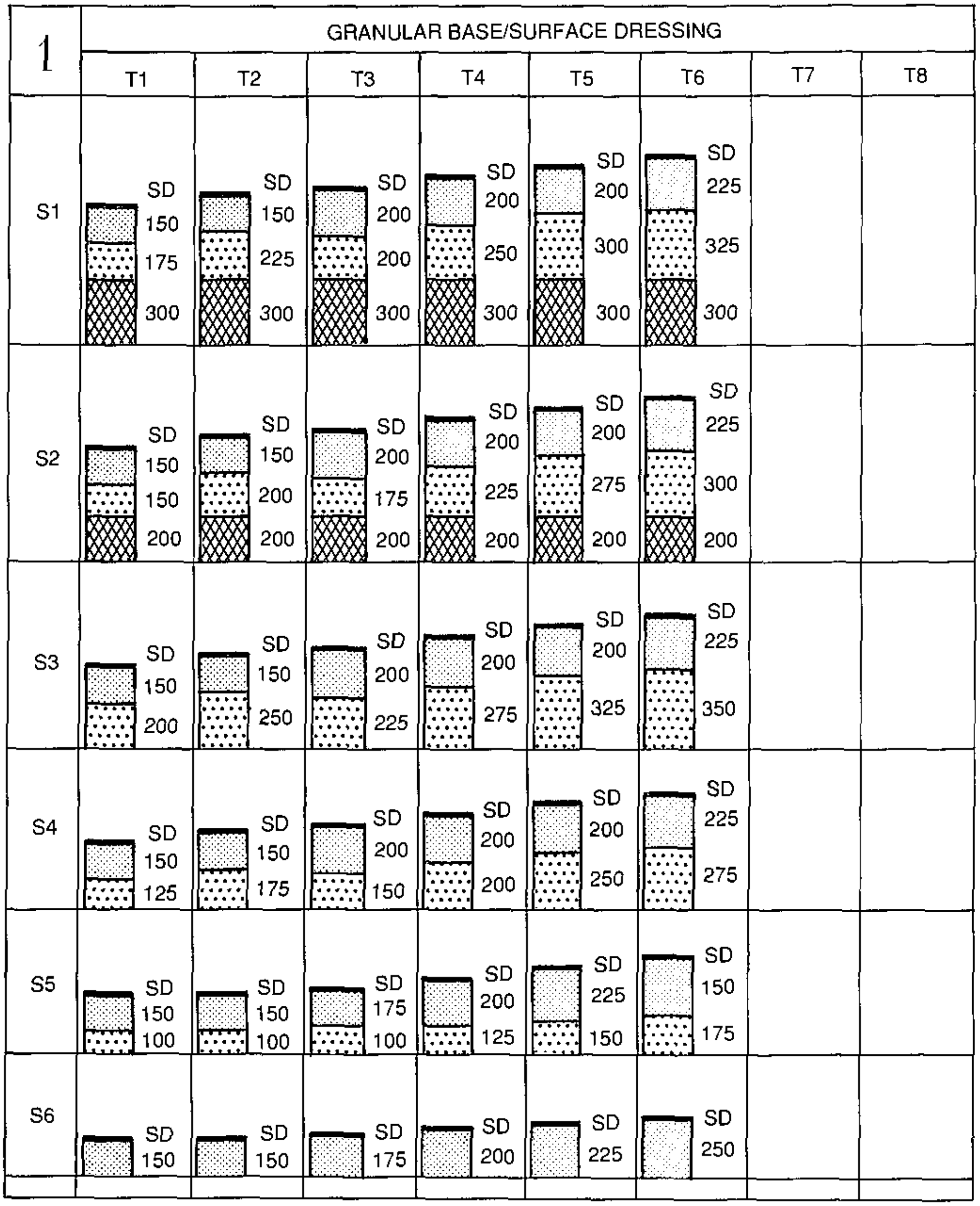


Fig.9.15 Example of a structural design catalogue (from Road Note 31).

to vary the exponential and to use a value less than four, a value that is more appropriate in the dry environment prevailing over much of Australia. In estimating the performance of different forms of surfacing and base, design charts are provided in which the dominant distress mode of the components of the pavement are considered. The AASHTO design method (1986) relies heavily on the concept of structural numbers. A design structural number for the pavement is derived from estimates of the traffic loading and the resilient modulus of the soil subgrade. Tables and Figures are given to indicate the appropriate strength coefficients of different forms of sub-base, base and surfacing. Layer thickness is used as a multiplier and the pavements are designed upwards from the subgrade, each layer having a design structural number such that the cumulative total is equal to or greater than the required structural number of the completed pavement. The method recognises the inherent variability of road making materials and construction processes and provides a detailed statistical approach to take account of this variability. State highways departments are responsible for the design of pavements within State boundaries. Climatic environments vary considerably over the USA and it is left to the individual States to adapt the method to be appropriate to the local environment.

Clearly we are a long way from worldwide unity in methods of design for flexible pavements but there is some consistency, for instance, in deriving appropriate values for the support offered by soil subgrades and in defining the significance of traffic loading. The methods of defining the structural properties of surfacings, bases and sub-bases are moving towards consensus. There are some obvious gaps in current knowledge; for example, in defining more closely the effects of climatic environment on pavement strength and in relating the effects of weather and traffic loading on the durability of pavements in hot climates. The purpose of this chapter is to describe the mechanisms which operate to determine the performance of pavements so that engineers are able to select a method of pavement design appropriate to the conditions in which they are working and also make the judgements that are inevitably necessary in the design of pavements on particular roads.

[Throughout the writing of this book I have been chafing at the convention of dimensioning in millimetres. It gives quite a false impression of the accuracy with which it is possible to work in the design and construction of pavements. I hope that the sympathies of other civil engineers will be enlisted by the suggestion that dimensioning in millimetres be reserved for those items where fine precision is possible and that dimensioning in centimetres be reintroduced for such items as pavement thickness, where the practical limits of accuracy are, at best, ± 0.5 centimetres].

9.5 Rehabilitation and strengthening of flexible pavements

Flexible pavements are designed on the assumption that they will require strengthening at the end of their design life. If they have been correctly designed and properly maintained, this should be after some 15-25 years. In the meantime, one or more surface treatments will have been necessary to maintain the running surface against the actions of traffic and weather. The strengthening preserves the integrity of the pavement and much of the value of the original investment is thus retained. However, the design life will not be achieved if there are inadequacies in the design, the construction or in subsequent maintenance, and there is the further danger that the structural integrity of the pavement will be destroyed so that much of the value of the original investment is lost and additional heavy investment in rehabilitating the pavement is necessary.

9.5.1 Rehabilitation of pavements

The need for rehabilitation of a particular length of road pavement will often be obvious from inspection. The surfacing and base will be extensively pot-holed, there may be ruts in the wheel tracks, and it is likely that traffic speeds will be reduced by the unevenness of the pavement. Site investigation will be necessary to determine the depth to which the pavement is damaged and to identify the causes of the deterioration of the pavement. It may, for instance, lie in a deficiency in the arrangements to remove surface water and, in this case, the reconstruction should involve the installation of a drainage layer across the shoulder at sub-base or formation level as shown in Fig. 9.6. The pavement must then be redesigned following the design procedure for new pavements. If there is no sign of damage at sub-base level, it may be possible to obtain a good indication of design values for the CBR of the subgrade from in situ CBR tests.

If a source of weakness is identified in the base, it may not be necessary to remove the material. Natural gravel bases that are found to be excessively plastic can be stabilised in situ with cement or lime with the addition of sufficient new material to make up the thickness of base required. Graded crushed stone can be scarified, reshaped and recompacted to form the lower layers of the new base. One circumstance in which it is essential to remove material is when it is found to contain partially decomposed rock. If left in place, this is likely to break down still further to the clays which derive from the decomposition of some rock minerals.

In many developing countries there are lengths of road that have deteriorated to the extent that such major rehabilitation works are necessary. Although these tend to catch the headlines, in most road networks there is usually a high proportion of flexible pavement which is well designed and constructed and which can be kept in good condition by the much less expensive processes of surface dressing and the application of bituminous overlays when required.

The need for surface dressing is easily determined. In regions where rain occurs during the annual climatic cycle, surface dressing is needed as soon as it becomes evident that the existing surfacing is not fulfilling its function of preventing surface water from entering the roadbase. Any network of fine cracks appearing on the road surface is an indication of an urgent need to seal the road. In arid climates there is no such urgency but surface dressing may still be desirable to preserve a surfacing in danger of disintegration because of weathering of the bitumen.

It is less easy to decide when bituminous overlays are required. Some transport economists press hard for the use of surface profile to define suitable criterion. Measurements of surface roughness provide the means of justifying the works by the increased vehicle operating costs caused by the rough surface (see Section 6.8.4). Unfortunately, by the time that most pavements have deteriorated to the extent that vehicle operating costs are affected, it is far too late for a bituminous overlay to be effective. Such a pavement will be one more added to the list of those requiring major rehabilitation. In the computerised road management systems devised to indicate road maintenance and improvement needs, road roughness measurements can be useful in indicating those lengths of pavement in need of rehabilitation, and regular surveys of roughness over a road network can give an indication as to whether the road engineering effort is gaining or losing in the battle to build up an adequate road system. But integrated methods of measuring surface roughness are useless and can be quite misleading in determining when surface dressing and overlaying are needed to preserve the investment in individual road pavements.

9.5.2 Bituminous overlays

Bituminous overlays have two purposes. One is to restore a deteriorating road surface profile, typically one that has become rutted in the wheel tracks. The other is to strengthen the upper layers of pavement so that traffic stresses in the existing pavement are reduced, thus arresting any deterioration in the load distributing properties of the different pavement layers. The two forms of deterioration are not necessarily related. Rutting in the wheel tracks is usually most severe on roads with unbound bases, and the compaction associated with such rutting may well increase the strength of the pavement over the areas where it is most needed. Such rutting may also occur on very heavily trafficked roads with bituminous-bound bases. In contrast, bases bound with cement or lime may gradually lose their load spreading properties by fragmentation. In this case the purpose of overlaying is to delay the onset of this form of deterioration. This fragmentation will not be signalled by any deterioration in the road surface profile so it is necessary to have quite separate criteria for the correction of deformation in the wheel track and for the need to augment the overall strength of the road pavement.

The criterion for deformation in the wheel track is quite simple. It should not be allowed to develop to the extent that puddles of water persist on the surface after rain, nor to the extent that the ability to steer vehicles accurately is prejudiced. A commonly accepted criterion is that correction is needed on busy roads when deformation exceeds 5 millimetres under a 3-metre straight edge placed laterally across the road. On more lightly trafficked roads, higher levels of deformation can be accepted provided that the bituminous surfacing is not damaged to the extent that its impermeability to water is in jeopardy. The need for strengthening can be determined by measuring the overall dynamic modulus of the pavement, calculating the levels of stress developed in the different layers under traffic, and then comparing the results with the known capability of the materials in each layer to absorb stress without damage. Here we are back with the basic concept of flexible pavement design, namely that the dynamic modulus of each layer of pavement should be sufficient to reduce the stresses in the underlaying layers of pavement to a safe level i.e. to a level that will not result in permanent damage to the pavement.

Several methods of measuring the overall dynamic modulus of pavements have been developed. They can be conveniently separated into two main groups. In one, a moving load simulating road traffic is used and the vertical displacement of the road surfacing is measured as the load passes over the road surface. In the other method, falling or vibrating weights are used. Again, the vertical displacement of the road surface under load is measured, usually by transducers mounted on the road surface in an array at fixed distances from the loading apparatus. The results can be used to infer the dynamic stiffness of the different layers of road pavement.

In all methods, the surface deflections are proportional to the magnitude of the applied load but they can vary considerably with variations in the frequency of the applied loading. At higher frequencies, the stresses developed in the pavement have less time to relax before the next application of load. Dynamic stiffness therefore varies according to the frequency of the applied loadings. Stress relaxation is an important feature in the behaviour of bituminous mixtures and of wet subgrades. It is of less importance with cement and lime-bound materials and with dry soils. This means that there is no unique correlation between the results of tests at different frequencies. The AASHTO guide (1986) permits all types of dynamic deflection equipment to be used.

Fig.9.16 The Deflection Beam in use.

There are two main forms of testing which make use of rolling loads. One is the simple apparatus developed by A C Benkelman and known as the Benkelman beam or the deflection beam. This is a long pivoted beam which measures the deflection of the road surface at a point midway between the tyres of a twin-wheel assembly as it passes over the pavement. This apparatus is illustrated in use in Fig. 9.16. A two-axled truck is used with the rear axle load equally distributed between the two twin-wheel assemblies. Originally, a standard axle load of 6350 kilograms (14000 lb) was employed. Nowadays, in many countries an axle load of 8160 kg (18000 lb) is employed and loading up to 13000 kg has been reported. In quoting test results it is important to indicate the axle loading employed. The beam is made from a light aluminium alloy and the Overseas Unit of the TRRL has developed a form of beam that is not subject to distortion by the rapid changes in temperature that can occur when moving between sunshine and shade in hot climates (Smith and Jones (1980)). Tests are done in the wheel tracks and an expert team can undertake some 50 tests along the line of a road in one working day. Results are recorded manually and road surface temperatures are measured at each reading so that the results can be corrected for temperature changes.

For many years it was an ambition to automate both the deflection measuring process and the recording of results. This was successfully accomplished in France by the Laboratoire Central des Ponts et Chaussées in the mid-1960's with the Lacroix Deflectograph. This machine, with later modifications, is now in use in many countries for pavement testing and the design of

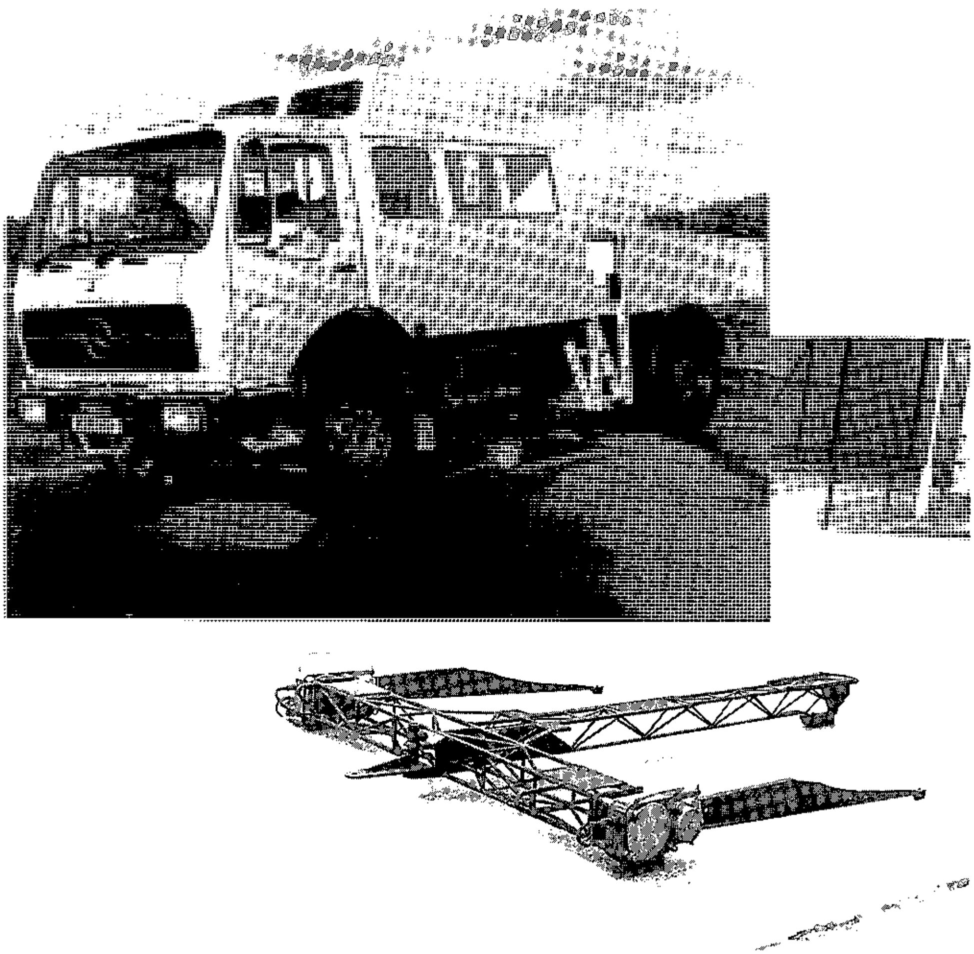


Fig.9.17 The Deflectograph.

pavement overlays. The Deflectograph, illustrated in Fig. 9.17, consists of a truck, a double deflection beam carried below the truck to register deflections in two adjacent wheel tracks, and recording apparatus.

The deflection beam is repeatedly winched forward between the tracks of the twin rear wheels and stopped on the road surface at intervals of 3.8 metres. The vehicle progresses forwards at a steady speed of about 2 km/hour and road surface deflections are recorded each time the rear wheels go past the contact point of the beam with the road surface. Each rear twin wheel assembly is loaded to 3175 kg (7000 lb). As with the deflection beam, higher loads can be employed.

Originally, pen recorders were used to record the deflections but nowadays digitised recording is more commonly used, giving outputs suitable for computer analysis. A manually operated event recorder is linked into the system enabling roadside features to be noted so that the record of deflections can be related to the length of road being surveyed.

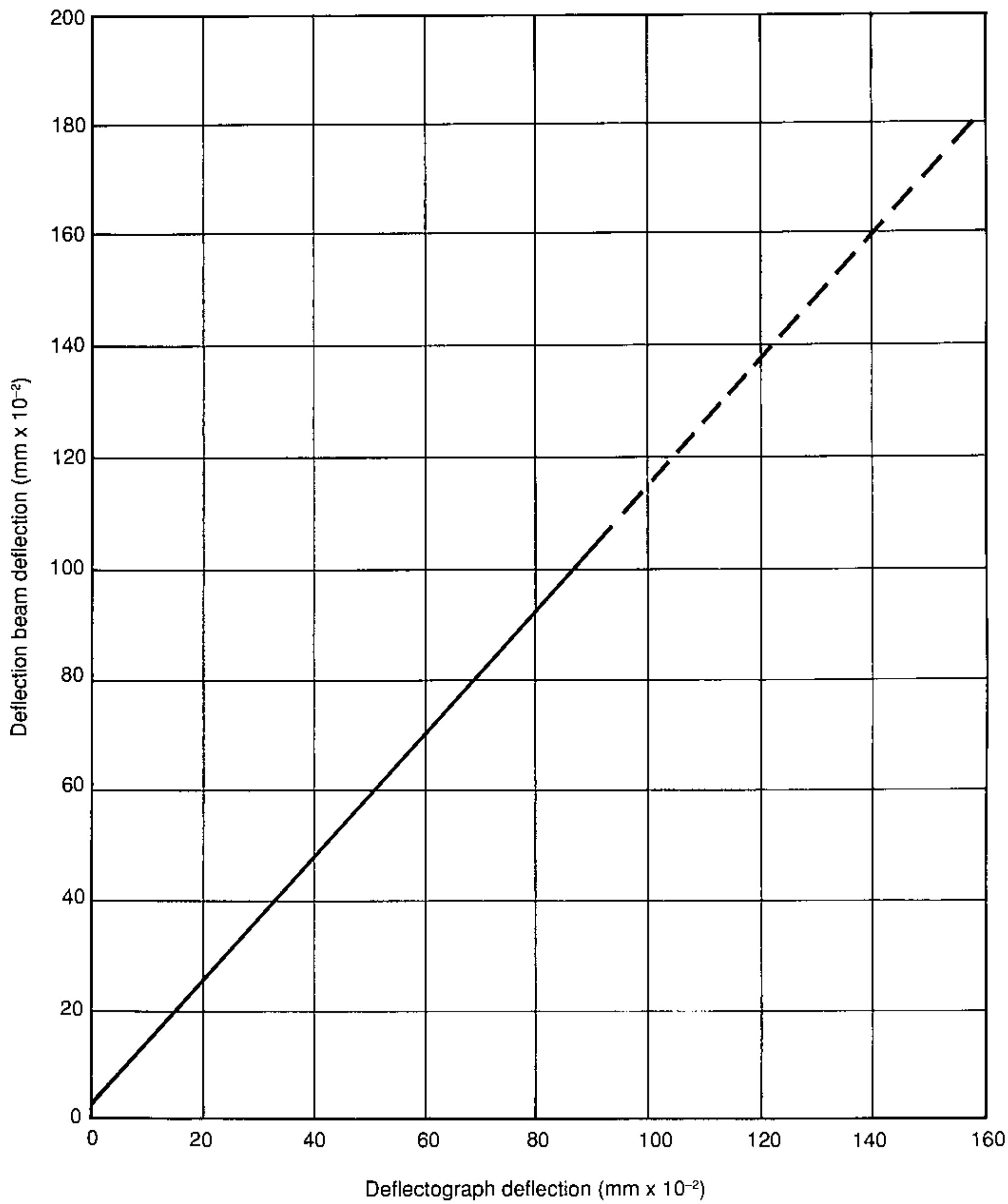


Fig.9.18 Correlation between Deflection Beam and Deflectograph.

The load is the same as that used with the Deflection beam but the duration of loading is somewhat less. Because of the close similarity between the loading systems, a unique correlation is available between the deflections measured by the two methods. This is shown in Fig. 9.18.

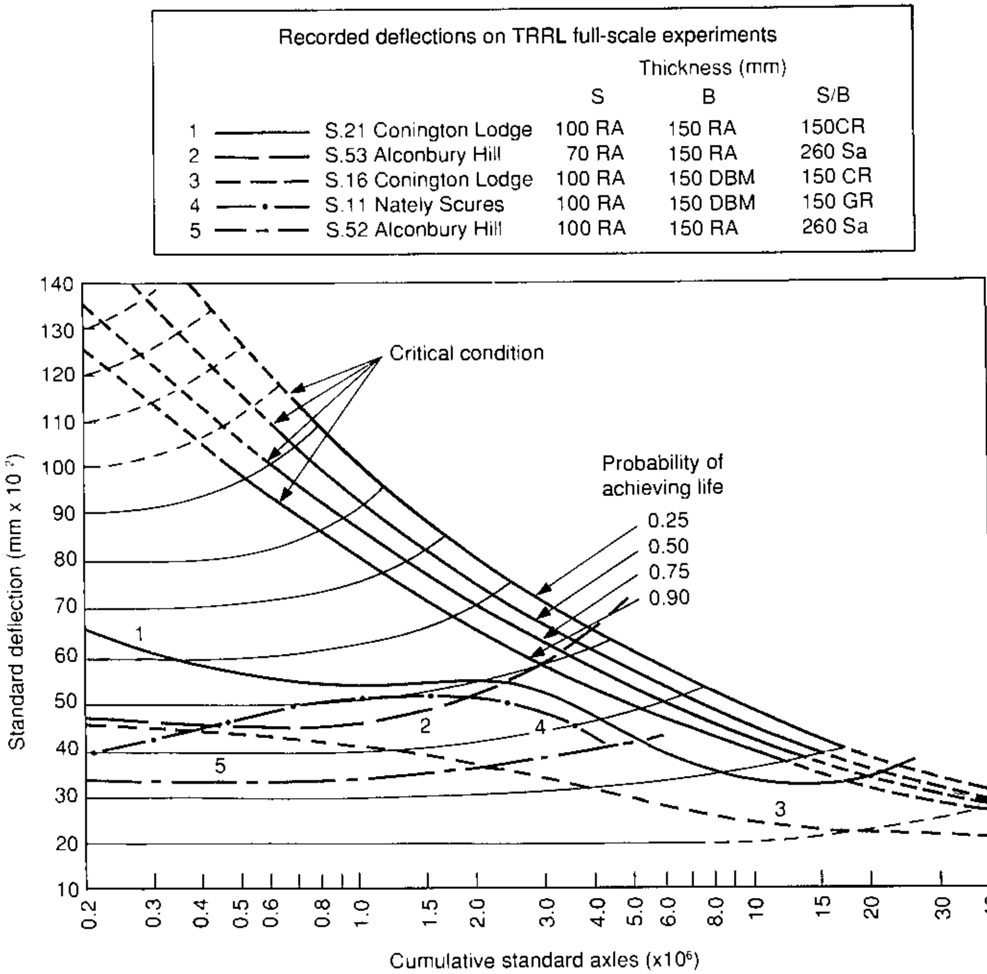


Fig.9.19 *Relationship between pavement deflection and life for flexible pavements with asphalt surfaces and roadbases (Kennedy and Lister, 1978 and Croney and Croney, 1991).*

Methods of using deflection measurements in overlay design have been described by Kennedy and Lister (1978). Fig. 9.19, from this paper, illustrates how deflection measurements can be used to indicate the expected design lives of flexible pavements. The curves all show a progressively increasing deflection towards the end of the design life, indicating a reducation in the dynamic modulus and the load spreading properties of the pavement. More recent work has indicated that the deflection history of flexible pavements is more complex. Superimposed on Fig 9.19 are the deflections recorded on pavements on five road experiments in Great Britain. Some of these show a persistent reduction in deflection with age. In this example, in which the pavements all have asphalt surfacings and bases, the reduction in deflection is probably associated with hardening of the bitumen caused by weathering which has, in turn, produced an effective increase in dynamic modulus. A similar reduction in deflection may occur on roads with unbound bases because of further compaction of the pavement under traffic.

An upturn in deflection readings is certainly an indication that a flexible pavement is beginning to deteriorate and that pavement strengthening by overlay should be put in hand. But it may not be practicable to keep even the main road network of a country under regular surveyance to record its deflection history. A more practical and useful application of regular deflection testing in any country lies in building up knowledge of the performance of the different forms of flexible construction in the local environment. This can be done by selecting particular newly built pavements on which good records are available of both design and construction and carrying out deflection surveys annually or, better still, at the end of any wet and dry seasons in the annual climatic cycles. Such regular deflection measurements should be done as part of the record of performance of pavements in any full scale road trials or experiments.

The other use for deflection testing lies in evaluating the thickness of overlay required on a particular length of road. Fig. 9.20 shows a typical overlay design chart indicating the thicknesses of bituminous overlay required in the United Kingdom for different levels of pavement deflection in relation to the extension of life required, the latter measured in terms of cumulative Equivalent Standard Axles. Fig. 9.21 shows a typical pattern of deflection record along a particular length of road. It shows, characteristically, that the length can be divided into sections in which different thicknesses of overlay are appropriate. The Figure also illustrates the variability inherent in the nature and performance of road pavements. Even with the most thorough care in survey, design

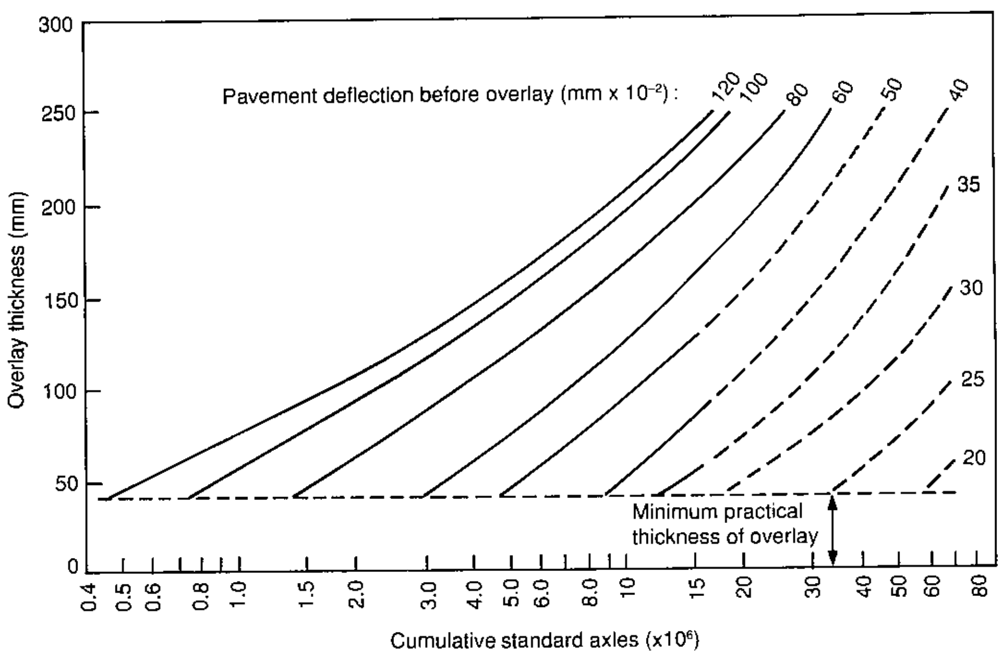


Fig.9.20 Overlay thickness design charts for road pavements with bituminous (asphalt) roadbases.

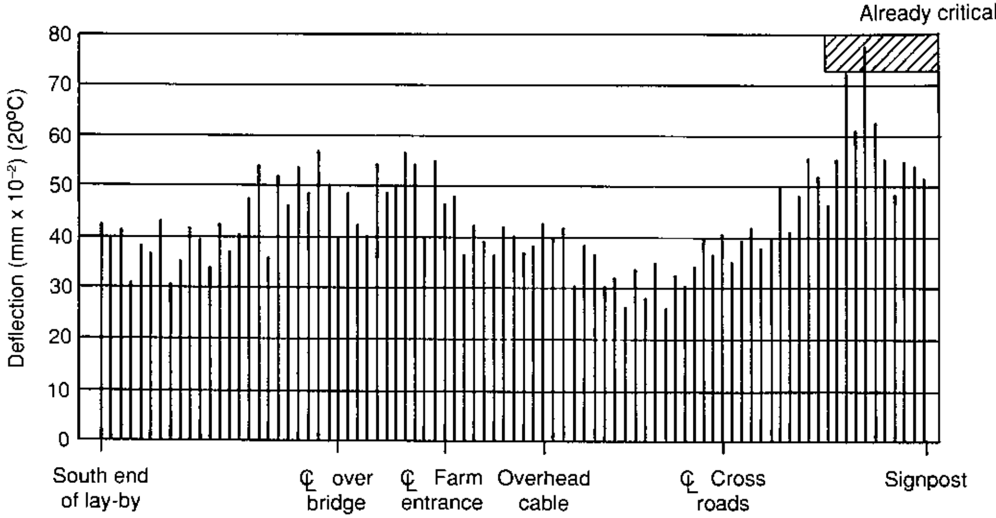


Fig.9.21 Suggested method of presenting the results of a deflection survey.

and construction, it is not possible to eliminate some variability, usually deriving from undetected variations in the support given by soil subgrades. Such variability can be turned to advantage in detecting lengths of road that are candidates for overlaying. The appearance of isolated surface crazing and potholes is an indication that the length of road is a candidate for a deflection survey to determine what action is needed to strengthen the pavement.

Some engineers make use of the shape of the depression bowl in the surfacing produced during deflection testing. A bowl with a relatively small radius of curvature suggests that weaknesses lie in the upper layers of pavement. A larger radius of curvature indicates that the layers with critical dynamic moduli are at lower levels in the pavement. This form of analysis is used, for example, in the NAASRA method of designing overlays in Australia.

More complex methods of interpreting the shape of the depression bowl to estimate the elastic moduli of pavement layers depend on the application of multilayer elastic theory, and the use of the results requires accurate theoretical or mechanistic models of road behaviour. These methods need careful empirical calibration to predict actual behaviour and are currently not sufficiently comprehensive to cater for the likely modes of behaviour under tropical conditions.

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10 Concrete pavements

10.1 Concrete roads in the tropics

As yet, concrete pavements have not been extensively used in most tropical countries. This is a pity because the maintenance effort required on concrete roads is less than that necessary to keep flexible pavements in serviceable condition. There are some tropical environments where concrete pavements have pronounced technical advantages; for example, over poorly consolidated soils and in climates where there is little variation in diurnal and annual temperatures so that provision for thermal strains in the concrete slabs is not of paramount importance.

It seems to be a characteristic of concrete pavements that either they prove to be extremely durable, lasting for many years with little attention, or they give trouble from the start, sometimes because of faults in design, but more often because of mistakes in construction. Misaligned dowel bars, for instance, can cause early trouble, and concrete of inadequate strength is likely to be broken up quite quickly under heavy traffic loads. This is not to say that the design and construction of concrete roads are more complex and difficult than for flexible pavements, but they are certainly different, and one reason for the infrequent use of concrete roads in developing countries is the lack of a corps of staff experienced in design and construction. In the Philippines, a brave and ambitious programme of rural road building with small local contractors was jeopardised by inadequacies in skill and supervision. On the other hand, concrete road building is firmly established in Chile, largely because of the initiative of local cement manufacturers in promoting the training of engineers and workmen in concrete technology.

Concrete is a rigid material, considerably stronger in compression than in tension, and the main objective in design is to make sure that the stresses imposed by traffic and induced by thermal expansion and contraction are contained within limits that the concrete can accept without fracturing. Concrete can be damaged by deleterious salts, either in the aggregate or entering the concrete in solution from an external source. But in the absence of such deleterious materials, concrete does not deteriorate. Indeed, its strength increases with age over the first few months. Unlike flexible pavements, concrete does not suffer deterioration from weathering. Neither its strength nor its stiffness are materially affected by temperature changes and this means that pavement design methods evolved from experience in temperate regions are applicable also in tropical climates.

In countries where vehicles are grossly overladen, this must be considered in the design. It is normally assumed that the fourth power relationship between axle loading and pavement damage applies. Concrete changes in volume with changing temperature and, to a lesser degree, with changes in moisture content. In concrete used as a surfacing, these changes in volume must be accommodated by the use of expansion and contraction joints, the spacings of which are determined by the range in diurnal and annual temperatures. In some tropical climates, notably in humid low lying areas near the equator, there are only small fluctuations in temperature and joints can be quite widely spaced. In other tropical climatic regimes, notably in deserts, there can be large fluctuations in both diurnal and annual temperatures and special attention is necessary to joint design and spacing.

For concrete to harden satisfactorily it must be well compacted and kept in a damp condition during the curing period. The initial setting of the cement is accelerated at high temperatures and this means that particular care is necessary in tropical climates to compact the concrete before this initial setting has developed. In dry climates, special measures are necessary to protect the concrete from the drying effects of sun and wind for at least seven days after it is laid.

10.2 Subgrades and sub-bases

The load bearing capacity of the soil subgrade is not important in the design of concrete pavements. The prime requirement is to provide a foundation on which construction traffic can operate without impairing the shape to which the surface of the foundation has been trimmed. Soils with soaked CBRs of 30 per cent and above are suitable, with one proviso. Water which enters the pavement through joints must have a path through which it can drain away. If water accumulates in the pavement under a joint, mud pumping is likely to occur as heavy vehicles pass from one slab to the next. If the subgrade soil is not free draining it is necessary to provide a sub-base. This sub-base should be free draining and should continue through the road shoulder as shown in Fig. 9.6. As an alternative, the upper 150 millimetres of the sub-base can be stabilised with cement or lime to produce a structured material resistant to mud pumping.

10.3 Concrete mixtures for road pavements

The main influences on the structural performance of concrete in roads are the strength of the concrete and its coefficient of thermal expansion. It is clearly an advantage to use aggregates which produce concrete with low coefficients of thermal expansion. Concretes made with limestone tend to have low coefficients of thermal expansion, sometimes less than half those of concretes made with other rocks (see Table 4.3). But the surface of concretes made with limestone coarse aggregate is likely to become polished under heavy traffic and be dangerously slippery when wet. In wet climates it is best to confine the use of limestone-concrete to more lightly trafficked roads. A somewhat elaborate way of using limestone on more heavily trafficked roads is to employ other aggregates for the concrete in the uppermost layer of the slab e.g. the concrete laid over any mesh reinforcement that is employed.

Stresses from traffic loading require consideration of the modulus of rupture of the concrete. As with most brittle materials, there is a linear relationship between the magnitude of the applied tensile stress and the logarithm of the number of applications of the stress required to produce fatigue failure. Studies reported by Kestler (1953) and by Galloway *et al.* (1979) have shown that the stress to cause failure after 10^5 applications of load is approximately 80 per cent of the modulus of rupture developed in concrete after 28 days curing.

Measurements of modulus of rupture are usually made on freely supported concrete beams loaded centrally or at the third points. Such testing involves elaborate apparatus, and the results can show quite wide scatter. For these reasons it has not been possible to adopt measurements of modulus of rupture for routine design and for use in specifications. Concrete strength is universally measured and specified in terms of compressive strength at the ages of 7 and 28 days. Typical compressive strengths specified for pavement concrete are 30 N/mm^2 at 7 days and 42 N/mm^2 at 28 days, with the tests being done on 150 millimetre cubes under standard conditions such as those prescribed in BS 1881, Part 116 (1983) or ASTM C918.

The relationship between compressive strength and modulus of rupture varies according to the shape of the aggregate particles. Concretes made with angular particles from crushed rock develop higher moduli of rupture than concretes of the same compressive strength made with smooth, rounded gravel particles. Early research reported in 'Concrete Roads; Design and Construction', (Road Research Laboratory (1955)), established relationships between compressive strength and modulus of rupture for concretes made with different aggregates. Typical relationships are,

$M = 0.36\ F^{0.7}$ (Crushed rock aggregate)
$M = 0.49\ F^{0.55}$ (Gravel aggregate)

where M = Modulus of rupture (N/mm^2) after 28 days
F = Compressive strength (N/mm^2) after 28 days

With concrete of compressive strength 42 N/mm^2 at 28 days, the corresponding moduli of rupture are 4.9 N/mm^2 with crushed rock aggregate and 3.8 N/mm^2 with gravel aggregate. The modulus of concretes made with gravel aggregate could be increased by using mixtures of higher compressive strength but there are practical difficulties in doing this, particularly in hot climates, and it is normal to accept that concretes made with gravel aggregates have somewhat shorter fatigue lives than concretes made with crushed rock.

The mechanical strength of aggregates for pavement concrete is similar to that required in bituminous bases (see Chapter 6) and special care is necessary in the tropics to avoid aggregates that are partially weathered.

The grading and particle shape of the aggregate are important. In the coarse fraction, flaky and elongated particles produce difficulties in compaction and surface finishing. Specifications should require that the proportions of flaky and elongated particles be kept below limits such as those prescribed in BS 812, Part 105 (1990). Concrete is frequently made with coarse and fine aggregate from separate sources which are recombined in appropriate proportions during mixing. It can also be made with an all-in aggregate e.g. crusher-run stone. Broad grading envelopes for these materials are given in Table 10.1. Within these limits, the gradings should be smooth and continuous, avoiding gap-graded mixtures.

Where the fine aggregate is natural sand, limitations of supply may make it necessary to use quite fine sands, but these produce mixtures of poor workability. In view of the rapid onset of hardening of concrete in hot climates, it is better to use sands with gradings near the coarse limits of the envelope.

To secure concrete of appropriate strength for use in road surfacings, the amount of cement

TABLE 10.1
BROAD LIMITS FOR AGGREGATE GRADING IN PAVEMENT CONCRETE

Sieve size (mm)	Percentage by mass of total aggregate passing sieve				
	Coarse aggregate		Fine aggregate	All-in aggregate	
	40mm down	*20mm down*		*40mm down*	*20mm down*
50	100			100	-
37.5	90-100	100		95-100	100
20	35-70	90-100		45-80	95-100
14	-	40-80			
10	10-40	30-60	100		
5	0-5	0-10	30-100	25-50	35-75
2.36			60-100		
1.18			30-100		
0.6			15-100	5-35	10-35
0.3			5-70		
0.15			0-10	0-8	0-8

needed will be of the order of 240 kilograms per cubic metre of compacted concrete. The precise cement content will be determined during design by compression testing at 7 and 28 days. These tests will also provide the information needed to specify the grading of the aggregate within closer limits. On large contracts, control during construction may involve analysis of the wet concrete to determine aggregate grading and cement content. The analysis method is described in BS 1881, Part 124. On smaller contracts, occasional checks on the grading of the aggregate should be done but it is more important to make sure that the apportioning gear on the concrete mixer is kept in good order and is effectively used.

A road engineer ought to wince whenever he hears the phrase "pouring the concrete". Although air entraining agents can be used to improve workability, concrete that is so fluid that it can be poured into place will lack the strength and durability necessary in road pavements. Water-cement ratios need to be controlled below 0.50 to produce durable concrete.

The traditional method of measuring the workability of concrete involves the use of the slump cone (Fig. 10.1). Samples of the wet concrete are tamped to fill the cone. The cone is then lifted vertically upwards and the concrete allowed to subside. The slump is the difference in millimetres between the height of the cone and the highest point of the slumped concrete. With the lean dry mixtures used in roads, the slump is quite small. The test is not very sensitive in distinguishing variations in the workability of such mixtures. The Compacting Factor Test (Fig. 10.2) is more sensitive. In this test the uppermost conical hopper is filled with fresh concrete. The quick release flap-door of the hopper is then opened and the concrete falls to fill and overflow the smaller central hopper. Its surface is levelled with a hand-float and the concrete is then allowed to fall into the bottom cylinder. The bulk density of the contents of the lower cylinder is then determined to measure the compaction produced by the energy input from the falling concrete. The tests are described in BS 1881, Parts 102 and 103. The Slump Test is a useful means of controlling the tendency of workers on small contracts to make the concrete easier to handle by adding too much water to the mix. The slump should not exceed 75 millimetres. On larger contracts a limiting Compacting Factor should be derived from the testing undertaken to design the mix proportions

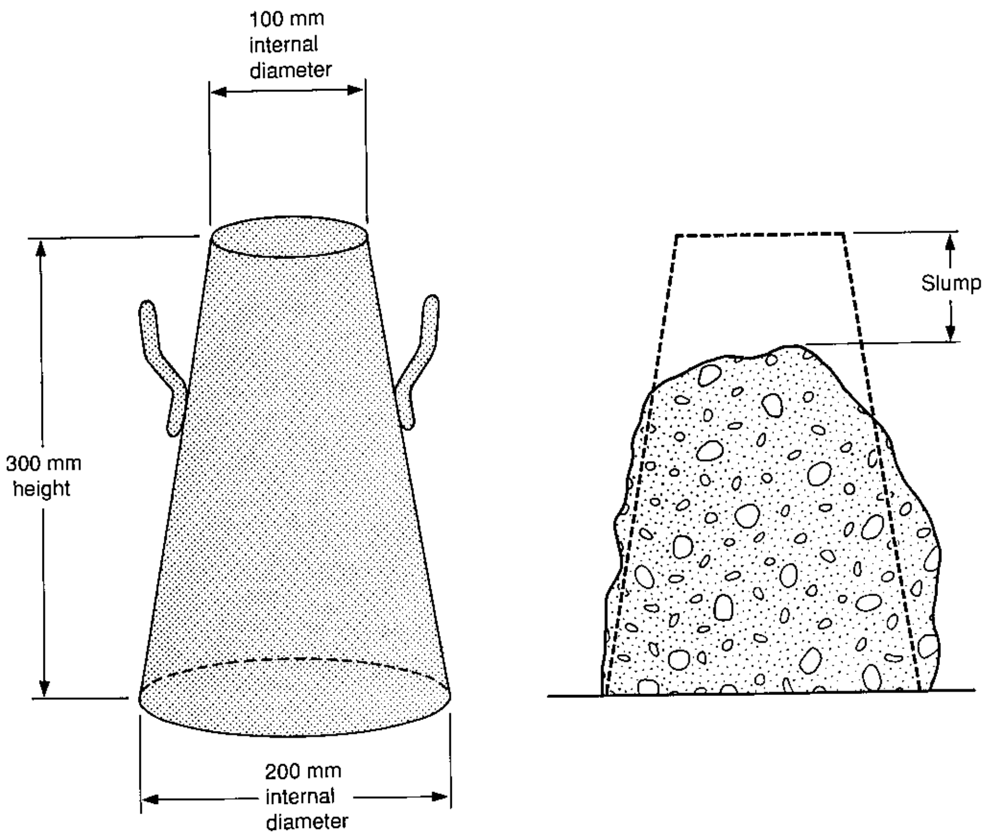

Fig.10.1 *The Slump Cone for measuring the workability of concrete.*

and should be used in the specifications. This testing will also define the water content of the mixture in terms of an upper limit on the water-cement ratio.

10.4 Compaction

With compaction of concrete there is a resemblance to the concept of optimum moisture content in the compaction of soils. A mixture which is too dry is difficult to compact. If water is present to excess it is impossible to achieve maximum density in the aggregate-cement mixture. Indeed, in a mixture which is too wet there is a danger that a slurry of water and cement will rise to the surface during compaction. This layer will later flake away exposing the weakened concrete below.

Once the mix proportions have been closely specified and the method of compaction defined,

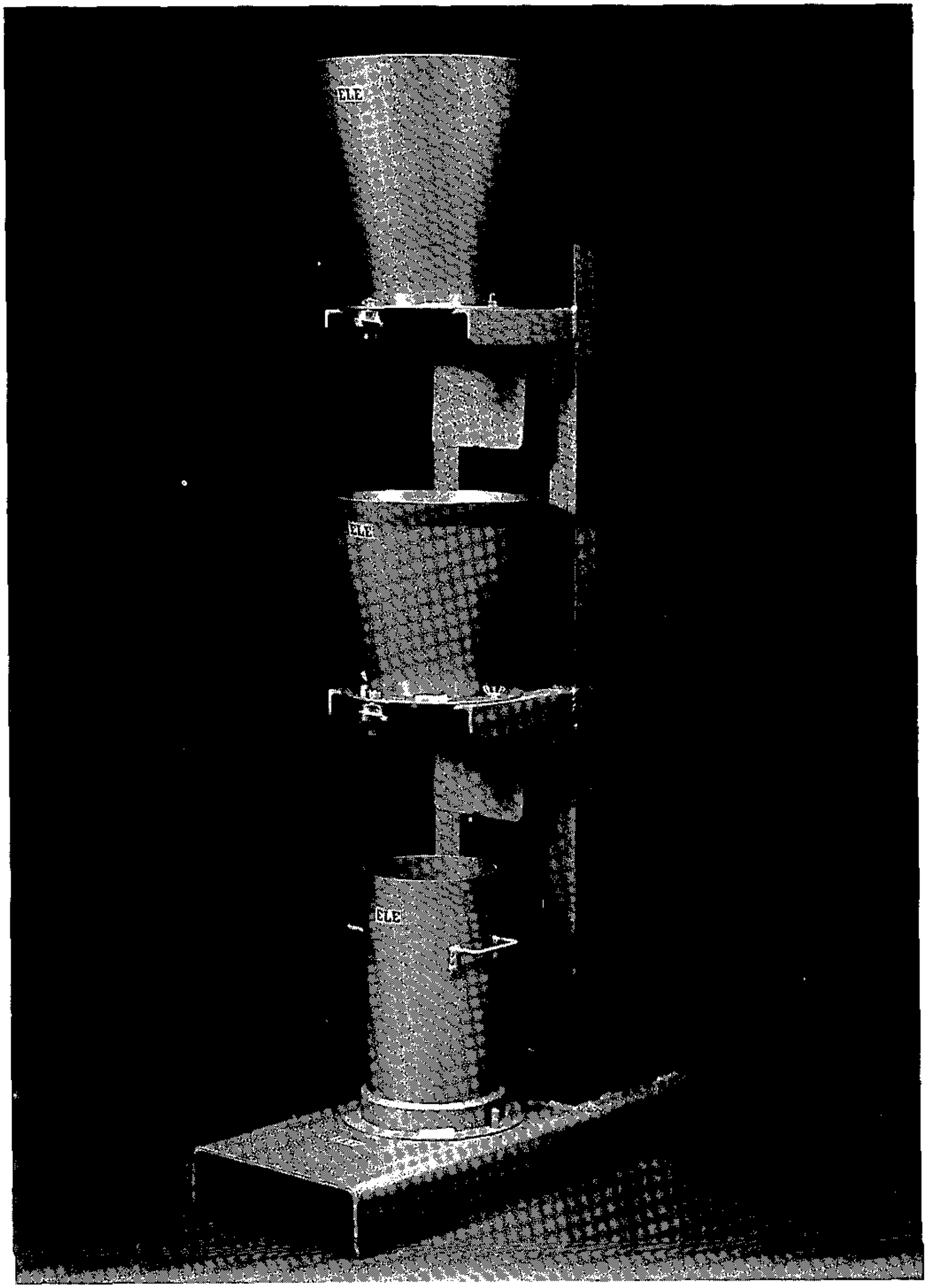

Fig.10.2 Apparatus for the Compacting Factor test.

there is normally no need to measure the densities achieved in the completed concrete. However, particularly with thicker slabs, segregation of the coarse and fine aggregate may occur during spreading of the concrete, with the coarser particles gravitating to the bottom of the slab. This risk is greatest with concrete that is too wet, and its occurrence can be detected by the inspection of cores cut from the hardened concrete. Many engineers prefer to use concrete made with aggregate no larger than 20 millimetres nominal size to minimise the risk of segregation.

10.5 Forms of concrete road construction

In concrete roads, the main structural element distributing traffic stresses over the soil below is the concrete slab. This slab may also provide the running surface for traffic, in which case, joints must be provided to relieve temperature stresses. Unreinforced concrete can be used, but on more heavily trafficked roads it is usual to employ reinforced concrete. This reinforcement is not intended to function as a structural element in the same way as the steel in a reinforced concrete beam or slab in a building. It is provided in the form of prefabricated mesh and is installed at mid-depth of the slab i.e. at the neutral axis. Its main purpose is to restrain the development of cracking in the concrete. Attempts have been made to produce rigorously designed reinforced concrete road slabs, even to introduce pre- or post-stressing of the reinforcement, but fixing the reinforcement where it can provide the most structural advantage has proved to be very complex and the expense is not justified by any gains in performance that might be made.

Continuously reinforced concrete is also used, normally as a road base on heavily trafficked roads where it can be covered with a bituminous surfacing sufficiently thick to reduce thermal stresses in the concrete below a critical level. Trials with continuously reinforced concrete as a surfacing without joints have not performed well. With no relief from the compressive stresses, 'blow ups' are likely to occur in hot weather.

Lean concrete is used as a road base, again with sufficient thickness of surfacing to reduce thermal stresses below a critical level. Indeed, there is now a progression of cement-bound mixtures ranging from normal concrete, through lean concrete and cement-bound granular bases, to cement-stabilised soil, all of which find use in building road bases.

10.6 Design of concrete pavements

10.6.1 Traffic

As with flexible pavements, cumulative Equivalent Standard Axles can be used as an indicator of design traffic loading (see Chapter 9). With concrete roads there must also be doubts about the applicability of the fourth power relationship between axle loading and pavement damage, especially when it comes to considering the effects of grossly overloaded vehicles. Their damaging effects vary considerably accordingly to prevailing temperatures. Diurnal temperature changes produce temperature gradients through the depth of the concrete slabs. Surface

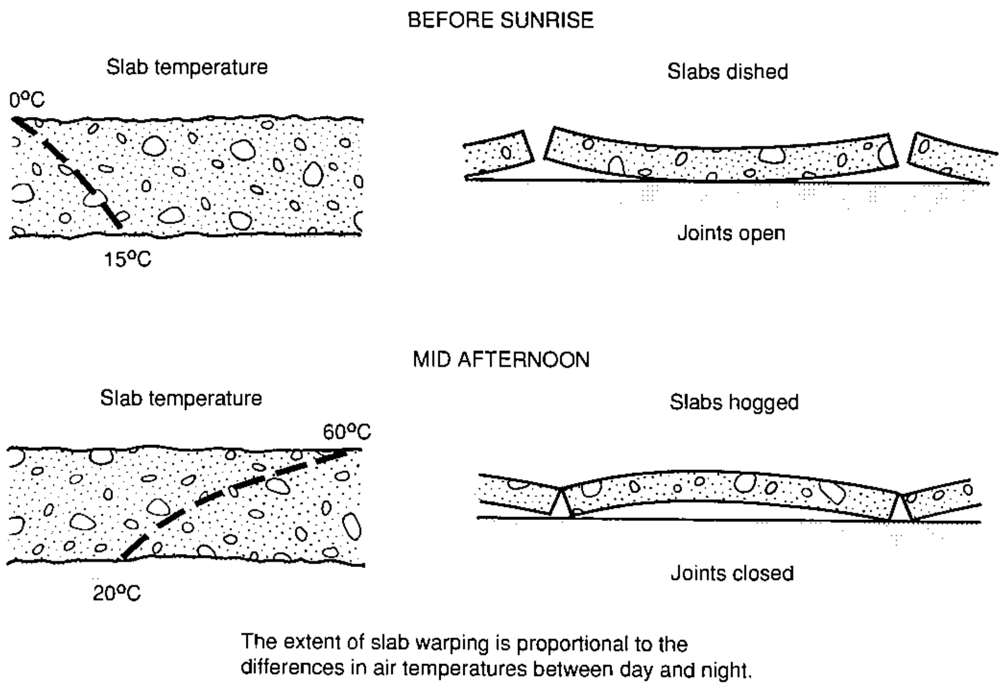

Fig.10.3 The effect of diurnal temperature changes on slab warping.

temperatures peak shortly after midday and are usually lowest in the early morning. Differential expansion causes the slabs to distort, hogging during the heat of the day and dishing when the temperature of the surface becomes lower than at the bottom of the slab. This effect is illustrated in Fig. 10.3. It is likely to be most severe in desert climates and occurs to some degree in all climates except those where there is little difference between day and night temperatures. The extent of such warping affects the performance of the pavement and must be taken into account in deciding joint spacing, thickness of slab, and choice of concrete strength. Roads carrying heavy vehicles at night are particularly vulnerable since the ends of the slabs then operate as unsupported cantilevers. Where traffic includes vehicles with axle loads of over 10 tonnes, special treatment is desirable to reduce the stresses imposed by this traffic. Such treatment may include:

i) Strong load transfer devices across joints.
ii) Closer joint spacing.
iii) Use of mesh reinforcement in the concrete.
iv) Increased slab thickness.

10.6.2. Failure criteria

The ultimate condition determining the life of a concrete pavement is deterioration in riding quality. Some years before this limiting condition is reached, cracks will have appeared in the concrete surface. It is the gradual spreading of these cracks that causes the deterioration in riding quality. On more lightly trafficked roads it is often possible to extend the life of the pavement by providing a bituminous surfacing to restore riding quality. On more heavily trafficked roads, the development of the cracks may be associated with breaking up of the concrete to the extent that it must either be removed or broken down in-place to provide what is, in effect, the equivalent of a crushed stone base.

Integrated methods of measuring surface roughness are of limited value in providing criteria for deterioration in riding quality since it is rare for concrete roads to deteriorate uniformly. Variations in the quality of the work, often due to different weather conditions at the time of construction, can produce quite wide differences in the performance of adjacent lengths of concrete. On the most affected lengths, deterioration can be measured by the amount of cracking, recording the length of narrow and wide cracks per square metre. The rate of development of cracking can thus be determined to provide an indication of the timing and nature of repair work necessary. On well designed and constructed concrete roads, the first evidence of cracking may not appear for twenty years or more, with progressive deterioration thereafter according to the intensity of traffic loading.

Faults in construction, e.g. misaligned dowel bars in joints, may cause early local areas of disintegration. Indeed, it is the joints which remain the greatest source of weakness in concrete roads and long-lived concrete roads are inevitably those with which particular care has been taken over the design and construction of the joints.

10.6.3 Slab thickness

The analysis of stresses in concrete pavements by multi-layer elastic theory is simpler than with flexible pavements. Westergard (1926) produced the first theoretical analysis of the stresses developed in concrete pavements and, since then, finite element methods have been developed to bring greater precision to the estimation of the stresses developed under different loading conditions, see Acum and Fox (1951) and Zienkiewicz (1967). Were these methods to be used to design concrete road slabs with the full rigour employed in structural concrete, their application would be relatively simple. They would indicate the slab thickness and reinforcement needed so that tensile stresses developed in the concrete never exceeded a critical level with an appropriate safety margin. With allowances for the damaging effects of many repetitions of rolling loads, such construction would be prohibitively expensive. Concrete road slabs are designed on the assumption that fatigue fractures will ultimately occur, with safeguards built into the design so that the effects of these fractures are contained, the roads continuing to give good service often for many years after the cracking has first become evident. The theoretical analyses remain useful in indicating whereabouts in the slabs special defences are needed against the effects of tensile fracture. But in producing a practical design method there can be no substitute for the information deriving from carefully observed behaviour of concrete roads in the field.

Such information is available from the AASHO Road Test and from full scale experimental

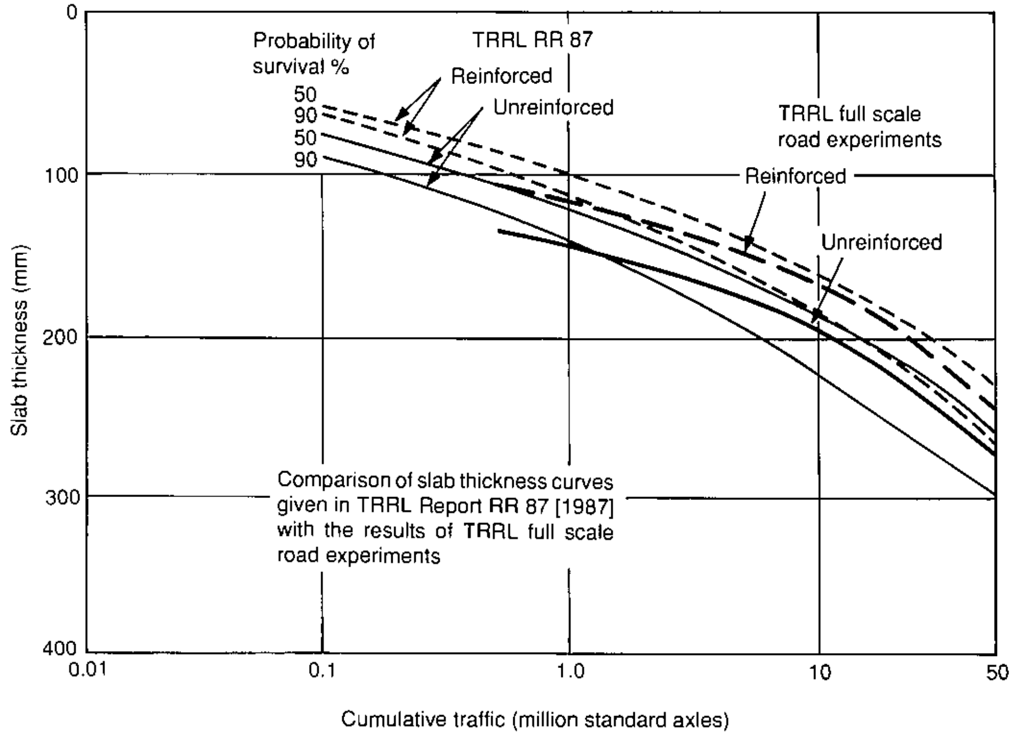

Fig.10.4 Design thickness charts for concrete road slabs.

lengths laid on public roads in Great Britain over the last 60 years. Croney and Croney (1991) have produced a detailed review of this experience. Figure 10.4, taken from this publication, illustrates the design curves relating slab thickness with cumulative standard axles that have been so derived. Although these curves extend to quite thin pavements for more lightly trafficked roads, practical considerations make it undesirable to attempt to construct concrete road slabs with a thickness less than 125 millimetres. The curves show the reduction in slab thickness made possible by the use of steel mesh reinforcement. Whilst unreinforced concrete is likely to be satisfactory in countries with small changes in diurnal and annual temperatures, reinforced concrete will be preferred in those countries where diurnal and annual temperature changes exceed some 20°C because of the need to contain the effects of thermal expansion and contraction of the concrete. Reinforcement is provided in rolls or in prefabricated mats and is normally laid on the fresh concrete after half the depth of the slab has been spread, and held in place by the further fresh concrete laid upon it. Care is therefore necessary to make sure that the reinforcement lies in a flat plane parallel with the surface. Heavier reinforcement is used on more heavily trafficked roads as shown in Fig. 10.5.

In climates where changes in slab temperature between day and night are extreme, thereby producing excessive warping of the concrete slabs, it is desirable to increase slab thicknesses by 25 millimetres. As already mentioned, special measures in slab design must be taken in countries

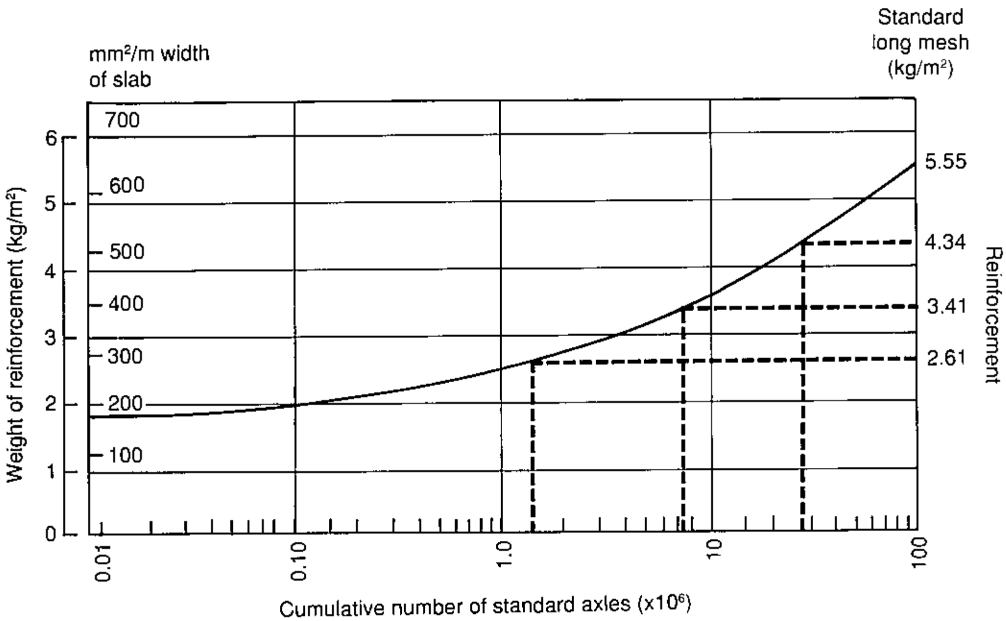

Fig.10.5 Relationship between traffic and weight of concrete slab reinforcement.

where axle loads are excessive. With these provisos, the differences in climate between Great Britain and tropical countries do not affect the applicability of these design curves. They have been incorporated in the general specifications issued by the Department of Transport in the United Kingdom, the current version of which is entitled 'Structural design of new road pavements', and was issued in 1987.

The design method for concrete road slabs most used in the USA is that issued by the Portland Cement Association. In its most recent form it has been described by Packard and Tayabji (1985). The method incorporates design charts requiring input on the flexural strength of the concrete, the characteristics of the soil subgrade, the cumulative axle loading together with a load safety factor, and the type of joint and shoulder. It also includes provision for considering the erosion of soil and sub-base material from under the slab and of shoulder material under the repetition of heavy axle loads. It is based on pavement stress and deflection data determined from a finite element computer program that models dowelled and aggregate interlock joints, and pavements with and without concrete shoulders.

10.6.4 Joints and joint spacing

The different types of joint are illustrated in Fig. 10.6. Expansion joints contain a joint filler which acts as a spacer during construction. It is made of compressible material which has some ability to recover after compression. Commercial joint fillers made of fibrous materials are available and

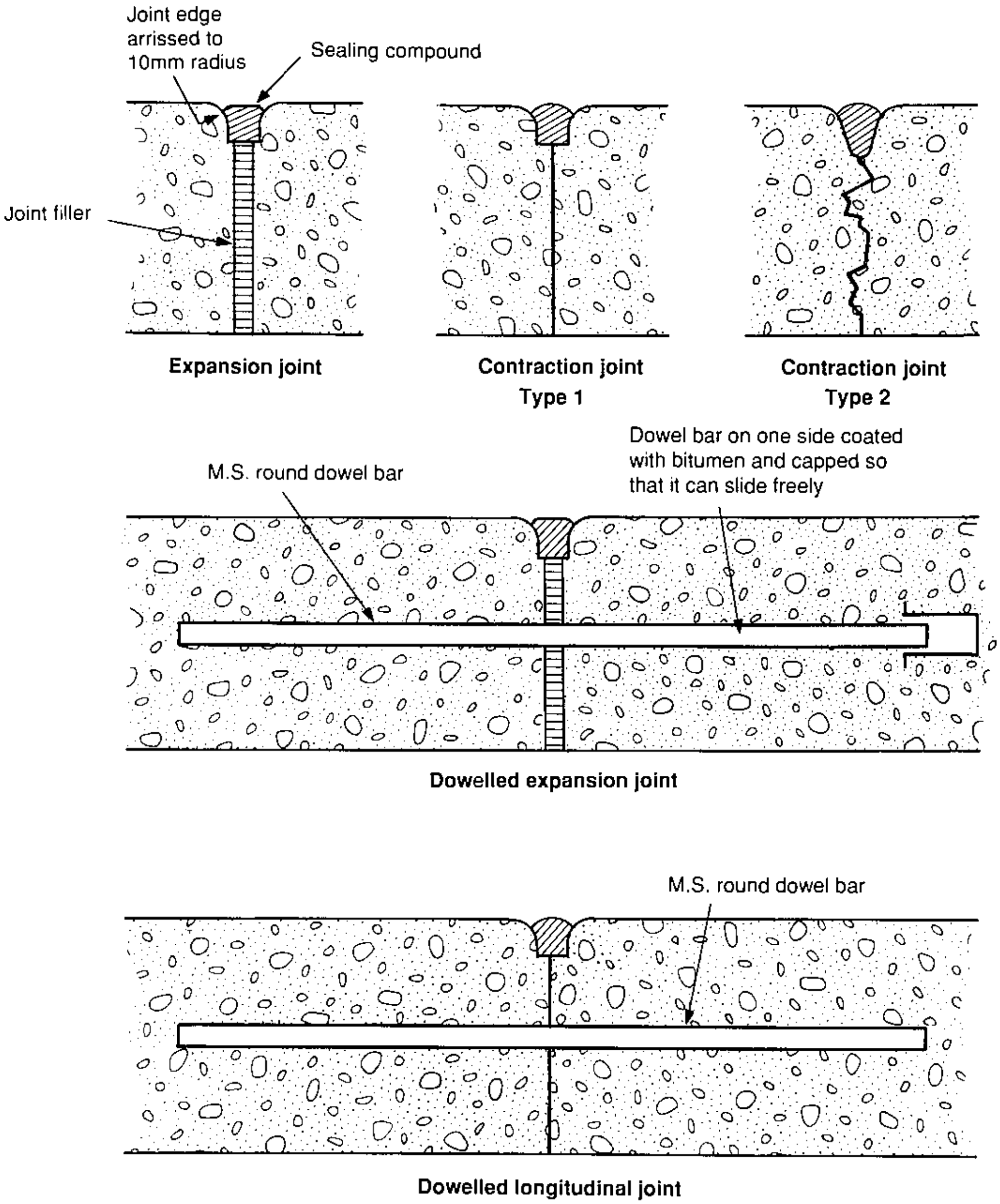

Fig.10.6 Joints in concrete pavements.

soft wood, free from knots, is also an effective joint filler except possibly in areas where it is likely to be eaten by white ants. Joint fillers are usually 15 millimetres in thickness.

All joints should be sealed. The sealing slots are usually 25 millimetres wide and the edges of the slots usually have a rounded arris. This arrissing must be done with care to avoid overworking which can bring a sandy laitance to the surface, weakening the concrete at a point where it is most vulnerable to damage. Specifications for testing commercial sealing compounds are given in BS 2499 and 5212 and in ASTM D1190 and 1191. As the concrete expands and contracts, these sealing compounds have to accommodate quite large strains. It is unlikely that they will remain effective in preventing water from entering the joint for more than two or three years. Nevertheless, as long as they remain in place, they fulfil their other important function which is to prevent loose stones on the road surface from being wedged in the joint. Stones, so wedged, can cause spalling of the edges of the joint as the concrete expands.

Type II contraction joints (Fig. 10.6) were developed with the advent of mechanised methods of spreading and finishing concrete roads. They consist of transverse slots penetrating to about one third of the depth of the slab. In effect, they are crack inducers. The raggedness of the crack that forms helps, in some measure, to transfer traffic loads across the joint. Originally, dummy joints of this type were made by mounting a strip of joint filler vertically on the formation, with the intention that the crack would form upwards to the surface. It is not easy to keep such strips in place as the spreader and finisher moves by. Nowadays, special equipment is available to cut slots in the concrete surface as soon as it has hardened sufficiently.

Another form of joint, not shown in Fig. 10.6, is the warping joint. Its purpose is to provide additional strength against heavy traffic loads on slabs that are warped by large fluctuations in daily temperatures. It resembles a dummy joint with fixed dowels placed in the concrete where the crack will form. Such warping joints act, in effect, as hinges. They may be useful in climates where annual temperature variations are not such that the full complement of expansion and contraction joints are required.

Experience has shown that, in most climates, joints are needed at 5 metre spacing in order to inhibit the early formation of cracks, to relieve temperature stresses, and to accommodate the initial volume changes that take place in concrete as it hardens. In climates with very small changes in diurnal and annual temperatures and where there is no serious overloading, it is likely that joint spacing may be increased to 7 metres or so provided that the lean dry mixtures required in pavement concrete are used. One advantage of such mixtures is that they minimise initial shrinkage of the concrete.

With concrete roads constructed in hot weather, contraction of the concrete predominates as the weather becomes cooler. In any subsequent expansion it is not likely that all the joints will be able to close to their original dimensions. For this reason, at least one joint in every four should be an expansion joint. With work done in the cold season, all joints should be expansion joints.

In concrete spread and compacted by hand, there is a practical advantage in using expansion joints with a joint filler of softwood. A wooden lath, 25 millimetres by 25 millimetres, can be nailed to the top of the joint filler to be used as a guide in arrissing the edges of the joint. After the concrete has hardened, the lath is removed to make the slot for the sealing compound. Removal of the lath is facilitated by making its cross section slightly tapered.

10.6.5 Load transfer at joints.

On roads carrying heavy vehicles it is desirable to provide dowels across joints to limit the vertical movement between slabs as vehicles pass over. It is also desirable to use dowels in roads over unconsolidated soils to prevent differential settlement between adjacent slabs. In longitudinal joints, these dowels are bonded into the concrete on both sides of the joint. In transverse joints, the dowels are bonded into the concrete on one side of the joint. Bonding on the other side is prevented, usually by coating the dowels with bitumen and, for expansion joints, by providing a loose end-cap as shown in Fig. 10.6. Dowels or ties in longitudinal joints are usually provided at intervals of one metre. On transverse joints they are more closely spaced, at intervals ranging between 300 and 400 millimetres.

Dowel bars range between 600 and 750 millimetres in length and between 20 and 30 millimetres in diameter with the heaviest bars being used on more heavily trafficked roads. It is particularly important that they are accurately aligned perpendicular to the face of transverse joints or parallel to the road if the joints are skewed. This is not difficult when spreading and compacting concrete by hand, particularly when alternate-bay construction is used. When spreading and compacting by machine, the dowel bars must be supported on robust cradles which remain in place in the concrete. Skill is needed, both in installing these cradles correctly and in locating where to install the sealing slot above them after the spreader and finisher has passed.

10.7 Construction

10.7.1 Laying the concrete

There are three levels of sophistication in the construction of concrete roads. At one extreme there are labour intensive methods employing a minimum of mechanical plant. In the middle comes the use of mechanical spreaders and finishers with the side-forms being placed to correct line and level by hand. At the other extreme there are slip-form pavers in which the side-forms are carried on the machine with wire guidance, or even lasers, to secure correct line and level. The increasing mechanisation has undoubtedly made high outputs possible. Slip-form pavers can lay over 5000 square metres of pavement in a day. They have made possible a consistently high standard of surface finish but have complicated the installation of joints and of reinforcement. Early failures are not uncommon when inexperienced staff are employed to carry out and supervise these operations.

In labour intensive works, the minimum plant required, in addition to transport for men and materials, is a rotating drum batch mixer and a vibrating tamper bar with an appropriate power source. Steel side-forms are an advantage but timber side-forms made by a local carpenter can be used. The coarse and fine aggregates are apportioned using wooden gauge-boxes, the proportions being set so that the correct cement content can be obtained by adding cement in bags and properly weighed half-bags. Water is apportioned by volume, with the slump cone in regular use to control workability. A gang of 12 men under a competent foreman can lay up to 500 square metres of concrete per day. In alternate-bay construction, a sequence of operations is established in which alternate bays are constructed on each successive day. This facilitates the installation of joints and the accurate location of dowel bars. In many developing countries there is a tradition of good workmanship amongst artisans. With encouragement, this tradition can be transferred to the laying of concrete roads by hand. In Zimbabwe, for instance, Parry (1990) reports the assembly of teams from a local contractor, initially unfamiliar with road concreting, who, under guidance, rapidly acquired the necessary skills, laying concrete roads that have every appearance of providing good service for many years.

The use of ready-mixed concrete is increasing all over the world and it is likely to be available in many larger towns in developing countries. The suppliers are habituated to the production of the somewhat wetter concrete mixes used in making reinforced concrete structures and therefore they are likely to need rigorous supervision to make sure that they supply concrete to the correct specification for use on roads. When ready-mixed concrete is used, the concrete should not be discharged directly from the drum of the delivery truck on to the formation. If the concrete falls heavily, it is likely to produce areas of densely compacted concrete that will be evident as raised areas in the finished surface. It should be discharged into a mobile hopper from which the concrete can be drawn off into chariot-wheel barrows of the type used in collecting the discharge from a rotating drum batch mixer. Skills have to be developed to rake the fresh concrete to an even profile. A surcharge of 10-15 per cent of the finished slab thickness is required to provide for the necessary compaction. Vibratory equipment is necessary for this compaction. Poker vibrators can be used but it is far better to employ vibrating screens working on the sideforms. These provide more uniform compaction and a more even surface profile.

The surcharge is regulated automatically when using mechanical spreaders and finishers. A modern train for the spreading, compaction and finishing of concrete is illustrated in Fig. 10.7. Such trains are used only by specialist contractors and a substantial investment in the building of new main roads is needed to justify their use. In most Third World countries, new distributor roads are needed, both in rural areas and in the rapidly expanding towns and cities. The need to provide gainful employment for the new city dwellers provides one good reason for using hand labour to build concrete roads.

Slip-form pavers are a remarkable example of the ingenuity of machinery manufacturers but they need an enormous effort in planning and operating the logistics to build roads at the rate of 1.5 kilometres per day which is within their capability. Even in industrialised countries, the rate of new road building is such that the machines can stand idle for much of the time. One consequence is that when they are brought into use, it is frequently necessary to mobilise new and inexperienced teams to operate them.

During the last decade there has been a remarkable development in the use of roller-compacted concrete pavements. Mechanical spreaders as used for bituminous materials are employed and sometimes adapted to increase the compacting effort of the vibrating screen. The compaction process is finished by using steel-wheeled vibrating rollers and joints are cut into the completed concrete. The concrete mixtures adopted are of low workability i.e. with particularly low water-cement ratios. One advantage of this development is that it employs equipment which has other uses in road making and it is claimed that costs are thereby much reduced. Starting in North America, the use of roller-compacted concrete has spread rapidly all over the world. A session devoted to this subject at the Sixth International Symposium on Concrete Roads (Cembureau (1990)) brought papers from Argentina, Australia, Canada, Chile, Japan, Germany, Norway,

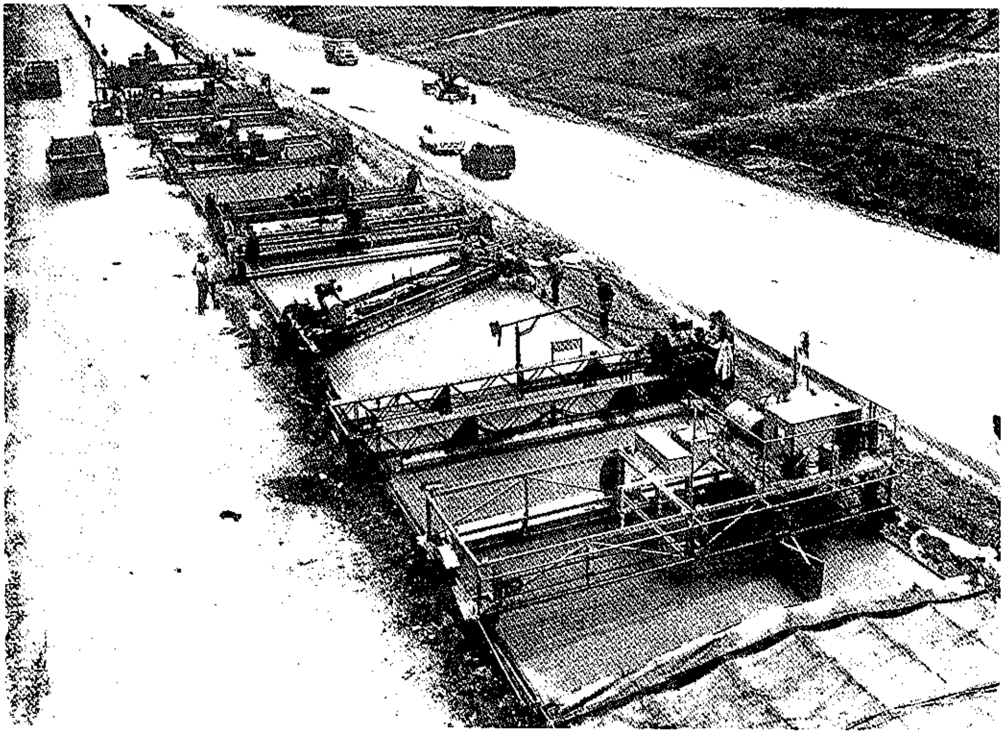

Fig.10.7 Spreading, compacting and finishing concrete pavements by machine.

Spain, South Africa, Sweden and the USA, all reporting experience in the last ten years. Roller-compacted concrete pavements are undoubtedly an important development in concrete road technology.

10.7.2 Surface finish

It is normal to produce a rugous surface by dragging stiff brooms transversely across the newly laid concrete, as illustrated in Fig. 10.8. Care is necessary to avoid damaging this broomed surface during the operations necessary to keep the concrete damp for the first 24 hours.

The very rugous surface required for an adequate resistance to skidding on busy roads in wet climates can be provided by cutting closely spaced grooves in the hardened concrete (see Fig. 10.9). Where speed in construction is a dominant consideration, these grooves may be cut in a longitudinal direction, but it is far more effective to cut them transversely so that water drains easily off the road surface. When this process was first used, the grooves were evenly spaced with the result that, under traffic, they produced a disconcerting high pitched hum at a single frequency. The spacing of the grooves is now randomised to reduce this effect. Grooved concrete roads are still rather noisy and are a subject of frequent complaint from the occupants of premises

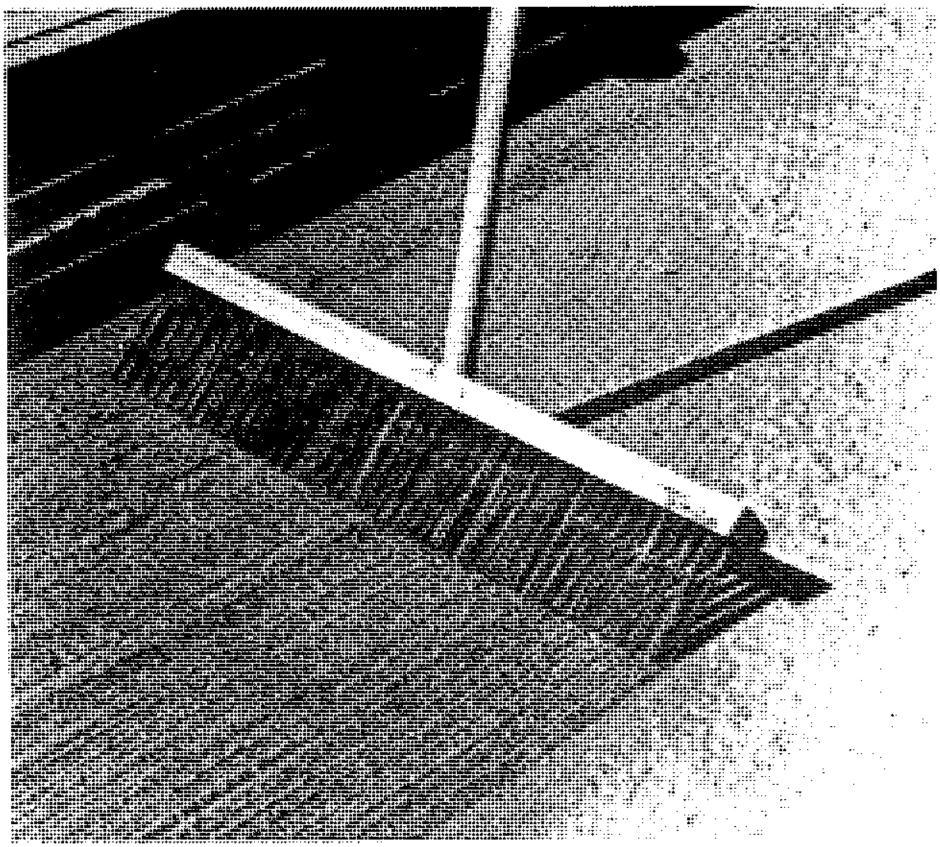

Fig.10.8 Broomed concrete to provide a rugous surface.

near busy roads. Concrete technologists are busy seeking methods of producing a non-skid surface with the same virtues as pervious bitumen macadam in reducing both traffic noise and traffic spray.

10.7.3 Concreting in hot weather.

High temperatures increase the rate at which cement hydrates. This produces two difficulties in hot climates. The first is that the concrete begins to harden rapidly after mixing so that it can become difficult to spread and compact. The second is that the rapid early gain in strength can be accompanied by shrinkage and cracking of the concrete with the result that the subsequent gain in strength is much less than with concretes cured at lower temperatures. For concrete cured in damp conditions at 20°C, there is normally a gain of some 40 per cent in compressive strength between specimens tested at 7 and 28 days and a further small increase in strength thereafter. For

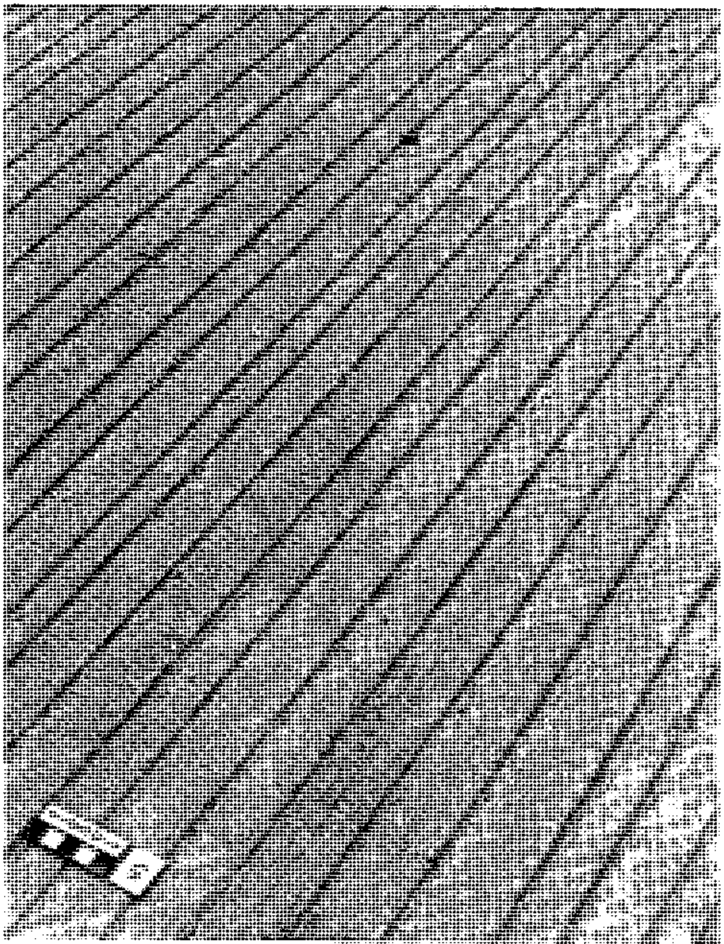

Fig.10.9 Grooved concrete road surface.

concrete cured at 50°C, there is likely to be little gain in strength between 7 days and 28 days with the prospect of continued weakening because of the shrinkage cracks that are likely to have developed during the rapid hydration of the cement. The modern trend to use more finely ground Portland cement has accentuated these effects.

Workability during the laying of concrete can be increased by adding more water to the mixture. To maintain the strength of the concrete at the required level it is then necessary to increase the cement content of the mix, but this has disadvantages, both in cost and in increasing the risk of shrinkage during hydration. Air entraining agents can be used to improve workability, and chemical agents are available to retard the setting of the concrete. Sugar has this effect. Such agents should be used only after preliminary testing has established their efficacy and confirmed that they have no harmful effects on the subsequent hardening of the concrete.

The most effective measures involve keeping the aggregate and mix-water as cool as possible before mixing, protecting the surface of the concrete from the effects of the sun and of drying winds for at least 24 hours after laying, and keeping the concrete damp for the first seven days.

Aggregate can be cooled by spraying with water and by protecting it from the direct rays of the sun. Water on site can be kept in containers with white outer surfaces to minimise the warming from solar radiation. With steel formwork, it is desirable to spray both formwork and the formation with water before the concrete is laid (but not to the extent that pools of water accumulate on the formation). It may be desirable, and a relief to the workmen, to start work at first light and break off for four hours or so in the heat of the day, resuming for two or three hours in the evening.

The surface of newly laid concrete can be kept damp by careful mist-spraying with water. After the initial set, which usually occurs within an hour of mixing, the surface can be covered with hessian, which is kept damp, or with light coloured plastic sheeting, or even with a dense cover of palm leaves. Portable shades can be used to protect each bay from the direct rays of the sun for 24 hours after it is laid. These shades should have skirts on each side to make sure that they do not direct drying winds over the fresh concrete. For the next week, water should be sprayed on the surface whenever the surface appears to be drying out.

10.8 Maintenance and repair

One of the outstanding virtues of concrete roads is that they require very little maintenance. It is impossible to keep joints sealed against the entry of water, particularly in climates with large diurnal and seasonal temperature changes, since the volume changes in the sealing slots are too large for the most accommodating sealing compound to absorb without parting from the concrete or fracturing. Sealing strips have been developed which are similar to those used in the joints of concrete reservoirs, but they do not survive for very long under the vertical displacements which occur at joints under traffic. It must be assumed that water will penetrate through joints and the construction must be such as to insure that the entry of water through joints does not weaken the construction (see Section 10.2).

Stones on the surface are a different matter. If they become wedged in the top of joints they can cause spalling of the concrete at the edges of joints. Stones are particularly likely to be present on roads carrying trucks containing aggregates and where the shoulder material contains large particles of aggregate that can be displaced on to the road pavement. The joints on concrete road should be inspected at least once a year, towards the end of periods of cooler weather, to remove any large particles of aggregate that are in danger of becoming wedged in joints. Thereafter, it may be necessary to provide fresh sealing compound.

Joints that have become badly spalled can be repaired by cutting out the defective concrete and replacing it with fresh concrete. This requires particular care and specialised skills as described by Mildenhall and Northcott (1986). Such work may be worthwhile when joint spalling occurs on fairly new concrete roads. On older roads it can be one of several signs of deterioration which mark the road as a candidate for overlaying.

The roughened surface obtained by brooming the newly laid concrete is likely to provide a high resistance to skidding for many years. Under very heavy traffic, the surface may wear to become unacceptably smooth.

One solution is to cut transverse grooves in the concrete as described by Salt (1982). But this is a tedious and rather expensive process and an alternative is to provide a bituminous surface dressing. A light coloured aggregate should be employed so that thermal movements are not accentuated and, as Hoban (1991) has described, the use of modified bitumens may help in obtaining a durable surface dressing under these somewhat arduous conditions.

Mud pumping may occur at the joints on more heavily trafficked roads. This is an indication of inadequacies in design i.e. that there has been no provision for water to be drained away from beneath joints and that adequate load transfer dowelling has not been provided. Deterioration can be arrested by drilling holes in the concrete slab and pumping in fresh concrete to provide support for the deficient foundation, as described by Mildenhall and Northcott (1986).

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11 Earthworks and drainage

11.1 Embankments

11.1.1 Introduction

Embankments are needed on all roads, providing culverts to permit cross drainage of surface water. Any road constitutes an interference with the natural pattern of overland water flow that has developed and care is necessary in design to make sure that this pattern is not damaged either by impounding water on one side of the road or by encouraging erosion in the side drains and where concentrations of water are discharged on to the surrounding countryside. Indeed, the construction of a road can often be made the occasion for works to control land erosion, particularly where land clearance adjacent to the road results in more rapid surface water run-off.

Major roads will be built to a selected design speed for traffic, involving adequate sight distances and limitations on longitudinal gradients. In undulating country, quite high embankments are likely to result. At the survey stage the quality of the soil in the cuts must be assessed to determine its suitability as filling, and every attempt should be made to balance the costs of cut and fill without borrowing material from surrounding areas. Minor roads will follow ground hugging alignments with the road on a shallow embankment normally built with soil excavated from the side drains. Higher embankments are likely to be needed on approaches to river crossings.

11.1.2 Embankment foundations

High embankments impose a considerable load on the soil below and, except when they are founded on rock, some compression is likely to occur in the foundations. Some transported soils are particularly vulnerable in this respect, wind-blown sands and unconsolidated estuarine soils being particular examples. In areas where wind-blown sands subject to collapse occur, it is unlikely that road building will involve the construction of large embankments. Where this is necessary, methods of inducing collapse similar to those used with other engineering structures can be used. These are reviewed in Chapter 3. Unconsolidated estuarine silty clays occur in many tropical countries and the special precautions necessary in building embankments over these soils are considered later in this chapter. Fortunately, the residual soils that are widespread in the tropics are usually not very compressible and such compression as does occur is likely to be substantially completed after the embankment is constructed. One possible exception lies in the halloysitic clays developing from volcanic ash. These soils can occur at quite low field densities and their somewhat fragile structure may be broken down causing collapse under the load of high embankments. In the soil survey preceding major road construction, note should be made of any soils encountered which, for any reason, might be subject to compression under load. A particular example lies in the quite narrow papyrus swamps which occur in water courses in Uganda.

It is particularly important at the survey stage to look for evidence of water flow during the wet season across the line of the road, either on the surface or at a shallow depth. Temporary perched water tables are a feature within many residual soils and, although these may not be obvious in the dry season, there will be indications of their presence in the vertical soil profile. There are also likely to be further clues in irregularities in the ground profile and in the vegetative cover. Drains must be installed to intercept such water flow and to discharge the flow away from the vicinity of the road. Useful notes on survey are contained in recommendations for the design of road embankments written by the National Institute for Transport and Road Research (1987 and 1978).

Fertile soils are a valuable commodity in tropical countries. Any fertile soil along the line of the road over the width of embankments and cuttings should be removed and stored for use in facing the side slopes of the finished road.

11.1.3 Soils for embankments

Almost all types of soil, from broken rock through to sandy clays, can be used for embankment construction, the main limitation being the ease with which they can be handled and compacted.

In the cold climates of northern latitudes, site operations are planned so that earthworks can be completed during the warmer weather. Even then, work may be suspended during periods of heavy rain and it may be necessary to cut potentially satisfactory soil to waste because of uncertainties as to how long it will take to dry out to a moisture content at which it can be satisfactorily compacted. Soils dry out more rapidly during fine weather in tropical climates. In regions with defined wet and dry seasons it is usual to plan operations so that earthworks can start as the wet season draws to a close. In hot climates where humid conditions prevail throughout the year it may well be necessary to find soil of low plasticity for use in building high embankments.

In main road construction it is usually cheapest to haul material from adjacent cuttings even though this may involve hauls of up to one kilometre. Soils of low plasticity are preferred since, with them, there is little difficulty in continuing work through periods of wet weather. With more plastic soils, assiduous care is necessary in wet weather to keep exposed soil surfaces shaped and compacted so that rainwater is shed to the sides. For embankments up to 6 metres high, the type of soil is of little consequence provided that it can be transported and compacted satisfactorily in place. With higher embankments it may be desirable to reserve material of low plasticity for the lower layers of the embankment. This may involve bringing rock from the lower layers of a cutting for use in the lower part of adjacent high embankments. With care in arranging the mass-haul strategy, double handling of the fill material can be minimised.

11.1.4 Stability of embankments

There are two aspects of embankment construction which may give rise to problems. One concerns the stability of the side slopes and the other involves the continuing and uneven settlement of the embankment. Shear tests followed by slip circle analysis may be undertaken on the soil to determine theoretical stable angles of repose but failures in side slopes are more usually

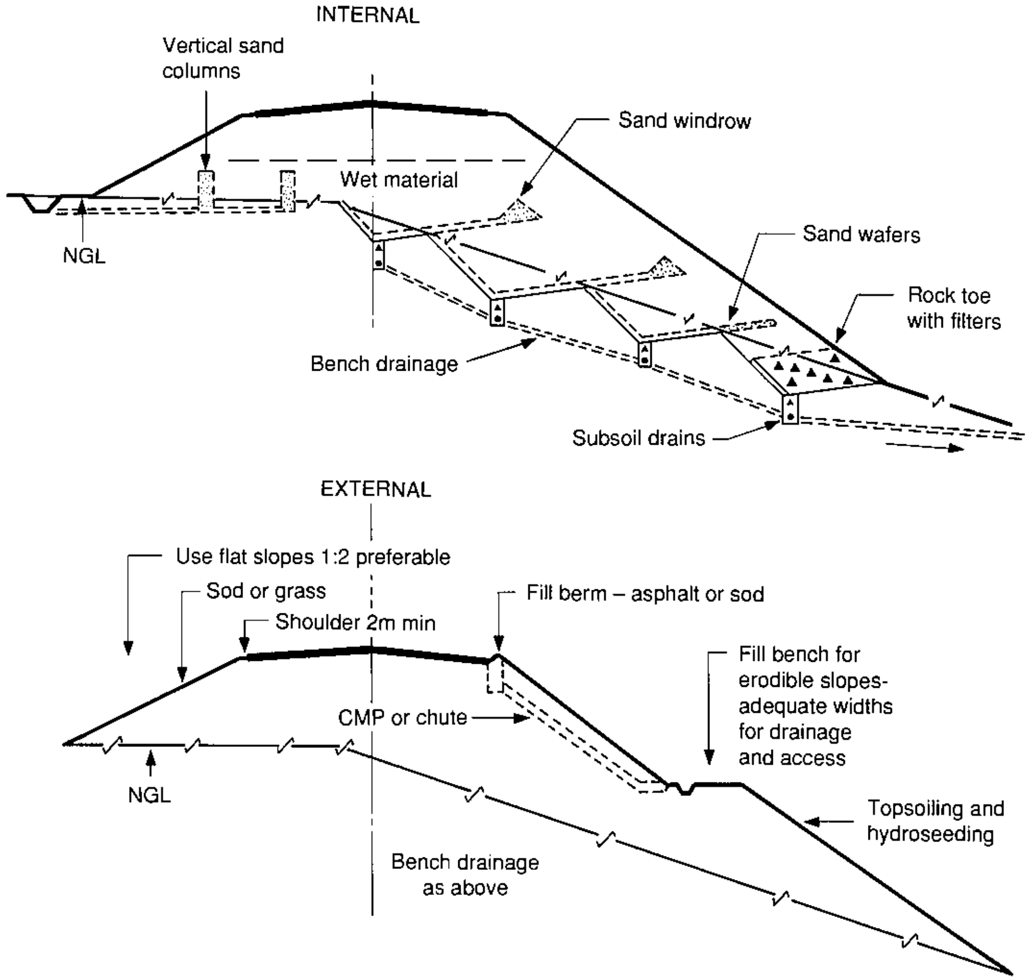


Fig.11.1 Drainage of embankments constructed with weak soils (CSIR, Technical Recommendations for Highways, TRH9).

irregular in form and are associated with faults in construction. It is usual to preserve a barrel-shaped cross section during construction to prevent rainwater from accumulating on the surface. The accidental inclusion of layers of weaker and more plastic soils can produce a slip plane. Drainage from the road surface may initiate the development of eroded runnels in the side slopes. On high embankments built with silty and clayey soils, it may be desirable to provide internal drainage systems and methods of disposing of surface water as illustrated in Fig. 11.1.

In monsoon climates, fluctuations in moisture conditions may produce dangers from volume changes in clayey soils, particularly in expansive soils containing montmorillonite. On low embankments it is an advantage to use shallow side slopes i.e. 1 in 3, and to cover these slopes with about 100 millimetres of granular material. The cracks which occur in black cotton soils in the dry season appear to develop in a direction at a right angle to the exposed surface. A longitudinal crack is likely to develop in the road surface during the first dry season after construction. It may also be observed that after deep-rooted grasses have withered on the surrounding ground as the dry season advances, they remain green on the roadside verges as they draw water from underneath the road. The longitudinal crack, usually near the verge-side wheel track, should be sealed before the oncoming rainy season. Thereafter, it is likely that the moisture regime under the road will have reached a stable condition and volume changes will be small. An additional precaution to reduce moisture changes in the soil under the road is to provide a bituminous surface dressing on the shoulders. The paramount consideration is to make sure that any cracks that appear in the road surfacing or near its edge are sealed promptly.

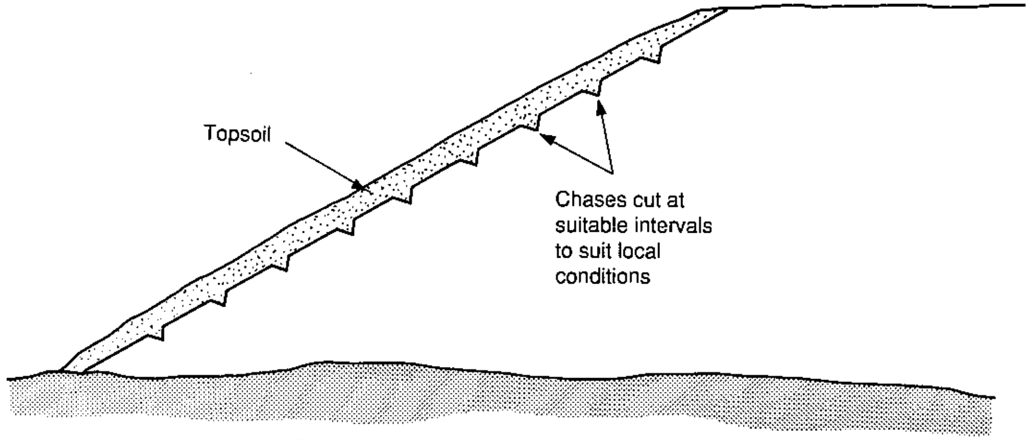
In areas where expansive soils predominate, the ground surface is likely to be flat, and high embankments will not be needed except at approaches to bridges over railways and river crossings. As Strongman (1963) has described, expansive soils can be used for such embankments provided that care is taken to compact them at an estimated equilibrium moisture content and the side slopes are covered with about 300 millimetres of crushed rock or other granular material to minimise seasonal moisture changes in the embankment soil.

A typical side slope for high embankments is between 1 in 1.5 and 1 in 2. Variants from this slope suitable for the local soils and climate are more reliably derived from local experience rather than from physical tests on the soils. Some generalisations are possible. It may, for instance, be desirable to use slacker slopes with silty and clayey soils, particularly in wet climates. Land values in urban areas may produce pressures for steeper slopes and, in these circumstances, advice from specialists in the use of proprietary fabrics and other forms of earth reinforcement can be sought. In all cases it is important to establish, as quickly as possible, some form of protection on the side slopes against the erosive forces of rain and wind.

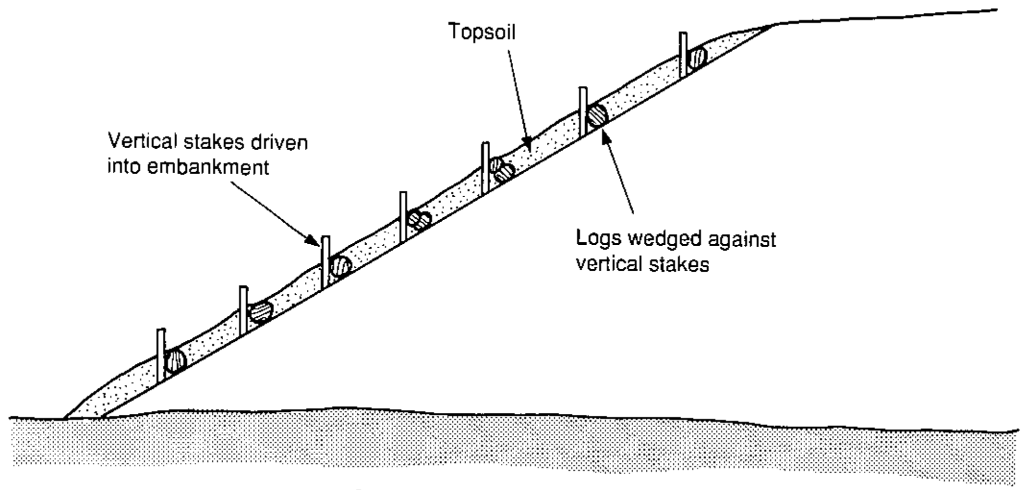
11.1.5 Slope protection

Some form of protection against rain and wind erosion is necessary on the faces of most embankments and cuttings. Most commonly, this is done by establishing a suitable cover of vegetation. Top soil must be provided as a cover and some method is likely to be needed to hold the soil in place until the vegetation has established a binding root system. Two methods are shown in Fig. 11.2. Sometimes the top soil will contain grass roots sufficient to regenerate an effective cover, but usually it will be necessary to provide this cover either by sprigging in young plants or by hydroseeding. This latter operation consists of spraying a mixture of suitable seed in a cellulose pulp over the surface. Local expert advice should be sought in choosing low-growing grasses or other plants that develop strong spreading root systems. In Ghana, for example, an indigenous form of portulaca has been used which spreads rapidly to form a dense cover, incidentally producing a carpet of bright purple flowers at the appropriate season.

In Nepal, the Overseas Unit of the Transport Research Laboratory has carried out an extensive series of experiments to assess the potential for vegetation to protect slopes against erosion and mass movement (Howell *et al*, (1991)). The studies have examined slope failure mechanisms, suitable species of vegetation, nursery planning, and planting configurations. Low cost engineering methods of slope support have also been evaluated.



CHASE METHOD FOR RETAINING TOPSOIL



STAKE AND BATTER METHOD

Fig.11.2 Methods of retaining topsoil on the slopes of embankments and cuttings (CSIR, Technical Recommendations for Highways, TRH9).

In circumstances where water may wash against the lower reaches of an embankment, such as at culvert headings and bridge abutments, special protection is likely to be needed. This can be provided by stone pitching, concrete blocks, or masonry. Below flood level it is desirable that this protection be firmly held in place by using gabions or in situ concrete. Such works should be regularly inspected and arrangements made for emergency repairs.

11.2 Compaction

11.2.1 Introduction

In soil embankments the purpose of compaction is to reduce settlement under the combined load of the superimposed soil, the pavement, and the traffic it will carry. In the pavement itself, and in the soil subgrade underneath the pavement, there is the further purpose to increase the stiffness of the materials so that they are better able to distribute traffic loads over the layers below. Uniformity of compaction is of prime importance to prevent uneven settlement. On long embankments uninterrupted by bridges and other structures, some settlement can be tolerated provided that it is uniform and does not prejudice the stability of the embankment. Where there are bridges and culverts, it is important to minimise settlement in order to preserve an even road profile. Special attention is always necessary to the filling behind abutments to bridges, care being taken to compact the soil so that future settlement is minimised and to provide drainage with suitable outlets down the buried face of the abutment.

Some tropical soils require special compaction techniques. With halloysitic soils, for instance, heavy compaction breaks down the soil structure reducing their natural permeability and strength (see Chapter 3). Montmorillonitic clays need special treatment to minimise the swelling and shrinkage associated with changes in moisture content. But, in general, the experience gained in the compaction of soils in temperate regions can be applied with equal relevance to the soils encountered in the tropics.

The differences in tropical climates, from arid, through monsoon, to perpetually humid, have a large effect on compaction techniques. In arid climates, shortage of water brings into prominence the importance of the techniques of dry compaction. In monsoon climates there is either too much water or too little and it is important to plan site operations so that major earthworks can be undertaken during dry seasons. In humid climates it may be necessary to seek out materials of low plasticity, for the construction of high embankments. Generally, the prevailing high temperatures mean that soils dry out much more rapidly than in more temperate climates. This can be an advantage when handling soils of high plasticity but it also means that much greater care is necessary in keeping the moisture content of the soil close to the optimum for maximum compaction with the particular compacting machinery that is being used.

In many tropical countries there are road vehicles in use with wheel loads much in excess of the limits that are permitted on roads in Europe and North America. These wheel loads produce a stress regime in the materials which is of the same order as that produced by the best compaction equipment at the time the roads are constructed. It is ironic that whilst the climate and materials available in tropical countries make it possible to produce lighter and cheaper pavements than in other parts of the world, this economy is prejudiced by the presence of grossly overloaded vehicles in the road traffic. One defence lies in compacting the soil subgrade and pavement layers as near as possible to refusal at the time of construction. The extra effort costs very little and, though it involves special vigilance in the control of site operations, it can be the means to minimise the damaging effects of overloaded vehicles (see also Chapter 9).

11.2.2 Compaction equipment

The ease with which soils can be compacted depends very much on their moisture content. In Figs 11.3 to 11.6 the performance of compaction equipment on four different soils is illustrated. The curves also show the results of standard and heavy compaction tests. All the curves show a peak, more or less pronounced, at which maximum density is secured. The accepted explanation for this peak is that increasing moisture content is providing the assistance of lubrication on the dry side, whilst on the wet side the increasing amounts of water displace increasing amounts of soil. The curves for the different compaction plant were obtained using the normal number of passes judged appropriate in the United Kingdom. With the soil on the dry side of the optima, further passes of the compaction equipment will produce higher densities. On the wet side there is increasing danger of overcompaction, particularly with the silty and clayey soils.

Two practical considerations follow from this. One is the importance of choosing the appropriate form of compaction equipment for a particular soil. The other concerns the need to build up experience with local soils to establish the best procedures with tbe compaction plant available. Table 11.1, taken from the 1976 edition of the Department of Transport's 'General specification for roads and bridge works', shows how such information is being codified in the United Kingdom. In later editions e.g. 1986, this information has been extended to cover a wider range of materials including the compaction of granular bases and sub-bases. For subgrades it is recommended that the number of passes of compaction equipment be twice that given in the Table. A comprehensive review of the compaction of soils and granular materials has been prepared by Parsons (1992).

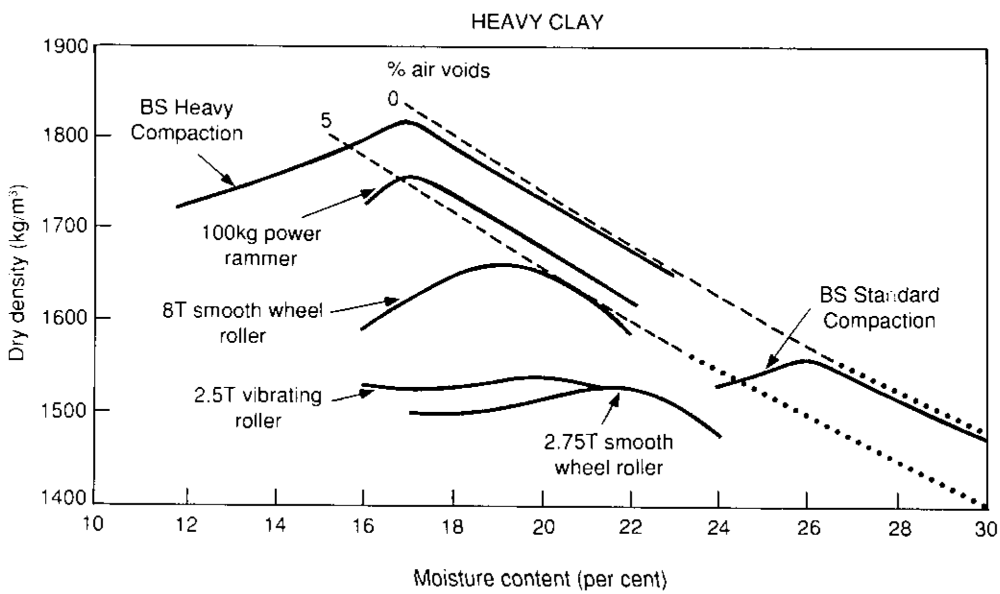


Fig.11.3 Compaction of a heavy clay with different rollers (Croney and Croney, 1991).

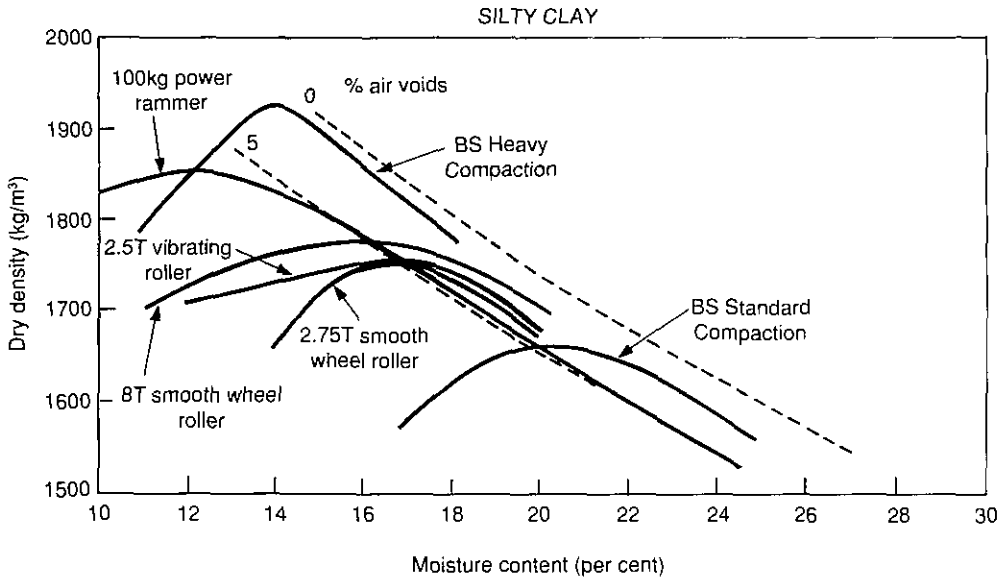


Fig.11.4 *Compaction of a silty clay with different rollers (Croney and Croney, 1991).*

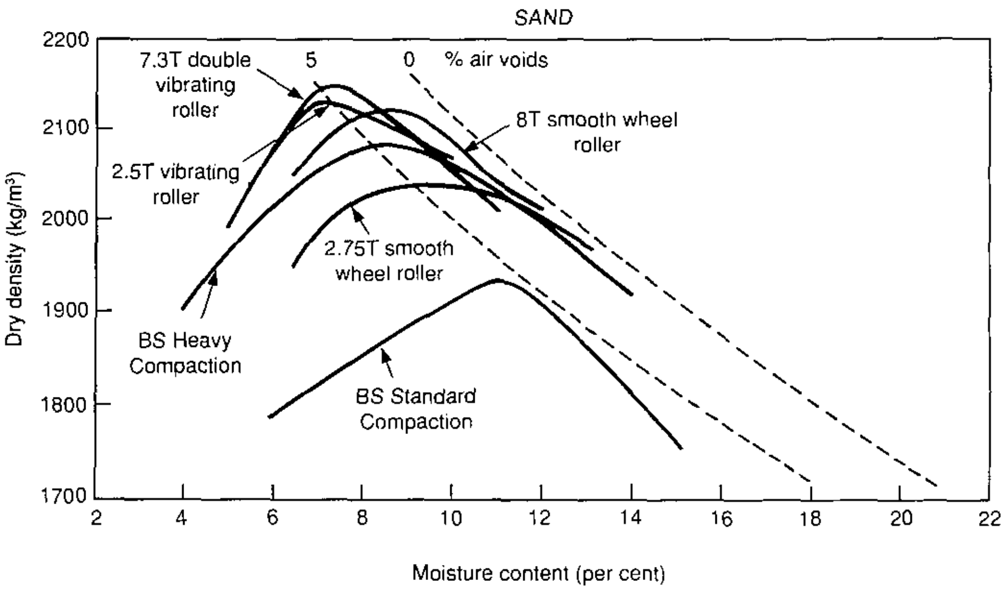


Fig.11.5 *Compaction of a sand soil with different rollers (Croney and Croney, 1991).*

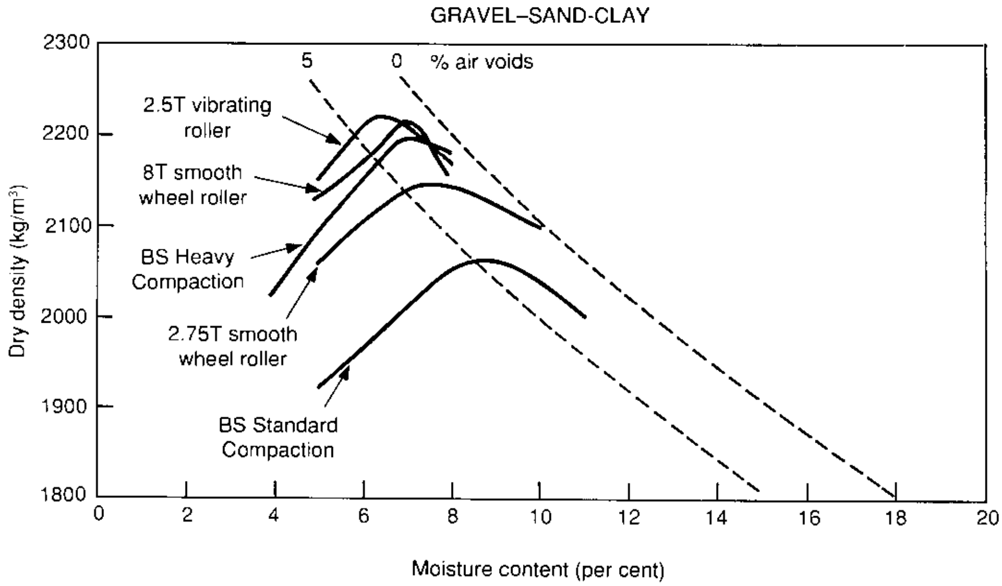


Fig.11.6 Compaction of a gravel-sand-clay with different rollers (Croney and Croney, 1991).

In the last two decades, many new models of compaction equipment have appeared on the commercial market with increasing use of vibratory compaction. This trend is likely to continue. They certainly incorporate useful technical advances including, for instance, the ability to vary the frequency and amplitude of the vibrations to take advantage of the resonant frequency of the soil mass. However, commercial claims can be exaggerated by enthusiastic sales staff. In developing countries, particularly, it is wise to consider what effort will be required to keep compaction plant in good working order and to look for compacting equipment that is useful over the range of local climatic conditions and soils.

11.2.3 Specifications and site control

The traditional methods of specifying the level of compaction to be achieved is by relative density i.e. by designating the percentage of maximum density obtained at optimum moisture content in the standard compaction tests. For filling in embankments it is usual to specify a relative density of 95-100 per cent of the maximum density obtained in the BS Light Compaction Test or its US equivalent, Proctor compaction. For the upper metre of soil subgrades, the required level is raised to 100-102 per cent. As Figs. 11.3-11.6 illustrate, these levels are usually not difficult to achieve and it is often possible and desirable to specify higher levels. The depth of each compacted layer is also usually specified. This varies according to the weight and efficiency of the compacting equipment and typical examples are given in Table 11.1. Higher values of relative compaction are required with granular bases and sub-bases, usually 98-100 per cent of the maximum density obtained in the BS Heavy Compaction Test or its US equivalent, Modified AASHTO. These

TABLE 11.1
COMPACTION REQUIREMENTS FOR EARTHWORKS

Compaction plant		Soil type					
Type	*Category*	Cohesive soils		Well graded granular and dry cohesive soil		Uniformly graded material	
		D	*N*	*D*	*N*	*D*	*N*
Smooth-wheeled roller	Mass per metre width of roll:						
	over 2100 kg up to 2700 kg	125	8	125	10	125	10*
	over 2700 kg up to 5400 kg	125	6	125	8	125	8*
	over 5400 kg	150	4	150	8	unsuitable	
Grid roller	over 2700 kg up to 5400 kg	150	10	unsuitable		150	10
	over 5400 kg up to 8000 kg	150	8	125	12	unsuitable	
	over 8000 kg	150	4	150	12	unsuitable	
Tamping roller	over 4000 kg	225	4	150	12	250	4
Pneumatic-tyred roller	Mass per wheel						
	over 1000 kg up to 1500 kg	125	6	unsuitable		150	10*
	over 1500 kg up to 2000 kg	150	5	unsuitable		unsuitable	
	over 2000 kg up to 2500 kg	175	4	125	12	unsuitable	
	over 2500 kg up to 4000 kg	225	4	125	10	unsuitable	
	over 4000 kg up to 6000 kg	300	4	125	10	unsuitable	
	over 6000 kg up to 8000 kg	350	4	150	8	unsuitable	
	over 8000 kg up to 12000 kg	400	4	150	8	unsuitable	
	over 12000 kg	450	4	175	6	unsuitable	
Vibrating roller	Mass per metre width of a vibrating roll						
	over 270 kg up to 450 kg	unsuitable		75	16	150	16
	over 450 kg up to 700 kg	unsuitable		75	12	150	12
	over 700 kg up to 1300 kg	100	12	125	12	150	6
	over 1300 kg up to 1800 kg	125	8	150	8	200	10*
	over 1800 kg up to 2300 kg	150	4	150	4	225	12*
	over 2300 kg up to 2900 kg	175	4	175	4	250	10*
	over 2900 kg up to 3600 kg	200	4	200	4	275	8*
	over 3600 kg up to 4300 kg	225	4	225	4	300	8*
	over 4300 kg up to 5000 kg	250	4	250	4	300	6*
	over 5000 kg	275	4	275	4	300	4*
Vibrating-plate compactor	Mass per unit area of base plate:						
	over 880 kg up to 1100 kg	unsuitable		unsuitable		75	6
	over 1100 kg up to 1200 kg	unsuitable		75	10	100	6
	over 1200 kg up to 1400 kg	unsuitable		75	6	150	6
	over 1400 kg up to 1800 kg	100	6	125	6	150	4
	over 1800 kg up to 2100 kg	150	6	150	5	200	4
	over 2100 kg	200	6	200	5	250	4
Vibro-tamper	Mass						
	over 50 kg up to 65 kg	100	3	100	3	150	3
	over 65 kg up to 75 kg	125	3	125	3	200	3
	over 75 kg	200	3	150	3	225	3
Power rammer	Mass:						
	110 kg up to 500 kg	150	4	150	6	unsuitable	
	over 500 kg	275	8	275	12	unsuitable	
Dropping-weight compactor	Mass of rammer over 500 kg						
	Height of drop:						
	over 1m up to 2m	600	4	600	8	450	8
	over 2m	600	2	600	4	unsuitable	

Notes: D = Maximum depth of compacted layer (mm)
N = Minimum number of passes
* Self-propelled rollers unsuitable. Rollers must be towed by track-laying tractors

criteria were evolved at a time when the choice of compaction equipment was limited. Nowadays, a wide choice enables equipment to be chosen which is suitable for particular soils and, apart from problems in controlling moisture content, there should be no difficulty in securing these levels of compaction.

On large works it is usual to construct trial embankments, basing the level of compaction specified on the results obtained with the different forms of compaction plant available. Here is another example of the value of the terrain evaluation procedures described in Chapter 2 which provide the opportunity to record information on the compaction characteristics of the soils of a region in a systematic way for use in the future.

The use of relative density for specifying the levels of compaction has the advantage common to all end product specifications namely that it indicates plainly to the contractor what he is required to do and leaves him with the responsibility of doing it. However, there are difficulties in making sure that the required levels of compaction are consistently achieved. The standard methods of measuring the field density of soils are not very precise. One reason for this lies in the intrinsic variability of most soils, but the tests are tedious and require considerable skill, particularly the Sand Replacement Test which is recommended for use with most soils. A careful study on one site revealed that over 40 tests were required to obtain a reliable indication of the range of compacted densities and to judge the extent to which the specification was being met in one area of fill. More alarming is the reproducibility of the tests. There have been numerous occasions when two teams of supposedly competent technicians have produced quite different ranges of results when measuring the compacted densities of gravel bases on the same site. Methods of measuring both density and moisture content using radioactive compounds offer a means of testing which is more rapid and much less tedious, but the difficulty of the intrinsic variability of most soils will remain. A more detailed description of soil density testing in the laboratory and in the field is given in Chapter 2.

Testing on the scale required to produce statistically reliable data is possible in evaluating the states of compaction achieved in trial embankments. Sufficient testing can be done on major works to indicate that compaction is at least approaching the level required but it is often impractical to undertake the quantity of tests necessary for the pass or fail criterion that is explicit in a contract specification. In the United Kingdom this has led to the widespread use of method specifications i.e. specifications in which the types of plant are specified together with the thicknesses of layer and the number of passes required with each particular item of plant (see Table 11.1). Site control is thereby greatly simplified, involving counting the number of passes and controlling the moisture content of the soil within specified limits. Contracts can provide for in situ density measurements to be made in sufficient quantity to ensure that the desired level of compaction is being maintained. A description of this method of specifying compaction is contained in the United Kingdom Department of Transport's 'General specification for road and bridge works'. With soil at a particular moisture content, the application of increasing compactive energy increases the strength of the soil to a density at which the soil is incapable of mobilising further strength at that moisture content. It is this phenomenon which is responsible for the fall off in strength and density with soils compacted at moisture contents above the optimum. Any additional energy will result in shearing of the soil. This is wasteful and can destroy any soil structure that is present in residual soils, producing a permanent weakening of the soil. An extreme example lies in the compaction of halloysitic clays but other clayey soils are vulnerable in this respect.

A field Moisture Condition Test developed by Parsons (1976) (see also Parsons and Tombs (1987)), which is used to determine the Moisture Condition Value (MCV), can be used as a rapid field method for determining the moisture content of soils without the need for oven drying. The test is also useful in assessing the suitability of cohesive soils for earthworks and in determining the trafficability of soils for vehicles and for civil engineering plant. The apparatus for this test is shown in Fig. 11.7. It consists of a falling rammer operating in a heavy steel mould. In this respect it is similar to the apparatus for standard compaction tests. The difference is that the rammer, of 97 millimetres diameter, operates within a mould of only slightly larger diameter and shearing of the soil is avoided by using a plastic sealing disc between the rammer and the soil in the mould. The penetration of the rammer at any given number of blows is compared with the penetration at four times that number of blows. Repetition of the test with different numbers of initial blows provides an indication of the compactability of the soil after different levels of energy input. The Moisture Condition Value is defined as 10 times the logarithm (to base 10) of the initial number of blows corresponding to a change in penetration of 5 millimetres. In the field, sufficient tests are done to construct a curve relating the initial number of blows to the subsequent change in penetration. An example is shown in Fig 11.8. With any particular soil there is a linear relationship between the MCV and the moisture content. This relationship can be established by tests on the soil over a range of moisture contents, as shown in Fig. 11.9. Measurements of the MCV of a soil can be simply and quickly obtained in the field and the moisture content can be derived from this linear relationship. The test and its use are described in the current revision of BS 1377 'Methods of test for soils for civil engineering purposes'.

11.2.4 Compaction at low moisture contents

When compacted over a range of moisture contents at a fixed level of compaction, all soils reveal an optimum moisture content at which maximum density is achieved. This is an advantage in temperate and humid climates because the optimum moisture content is usually quite close to the natural moisture content of the soil as it occurs in the field. In arid areas there are often difficulties in providing the large amounts of water needed to bring the soil to this optimum level. It is, as yet, not so widely recognised that many soils containing clay can be compacted to quite high densities at low moisture contents. Compaction curves for two such soils are shown in Figs 11.10 and 11.11. In Fig. 11.10 the compaction characteristics of an expansive silty clay from the Sudan are shown. Fig. 11.11 demonstrates the phenomenon with a clayey gravel from Kenya. In both cases the effects of the compaction on the CBR's of the soil are shown.

In both examples, laboratory examinations were followed by field trials which demonstrated the practicability of dry compaction (M O'Connell *et al* (1987) and C Ellis (1980)). Similar experience has been reported from Australia by Morris (1975). It seems likely that the explanation for this effect is that as the moisture content of the soil decreases below the normal optimum, increasing surface tension reduces the lubricating effect of the water. Further reductions in moisture content make it possible to break down the agglomerations of clay that have been held together by suction forces.

There are two further advantages beyond the elimination of the need for adding water. One is that it is possible to compact clayey soils at the naturally low moisture contents that prevail in arid areas. The use of imported water to produce a conventional optimum would result in subsequent drying out of the embankment soil involving changes in volume of the soil. The other is that it

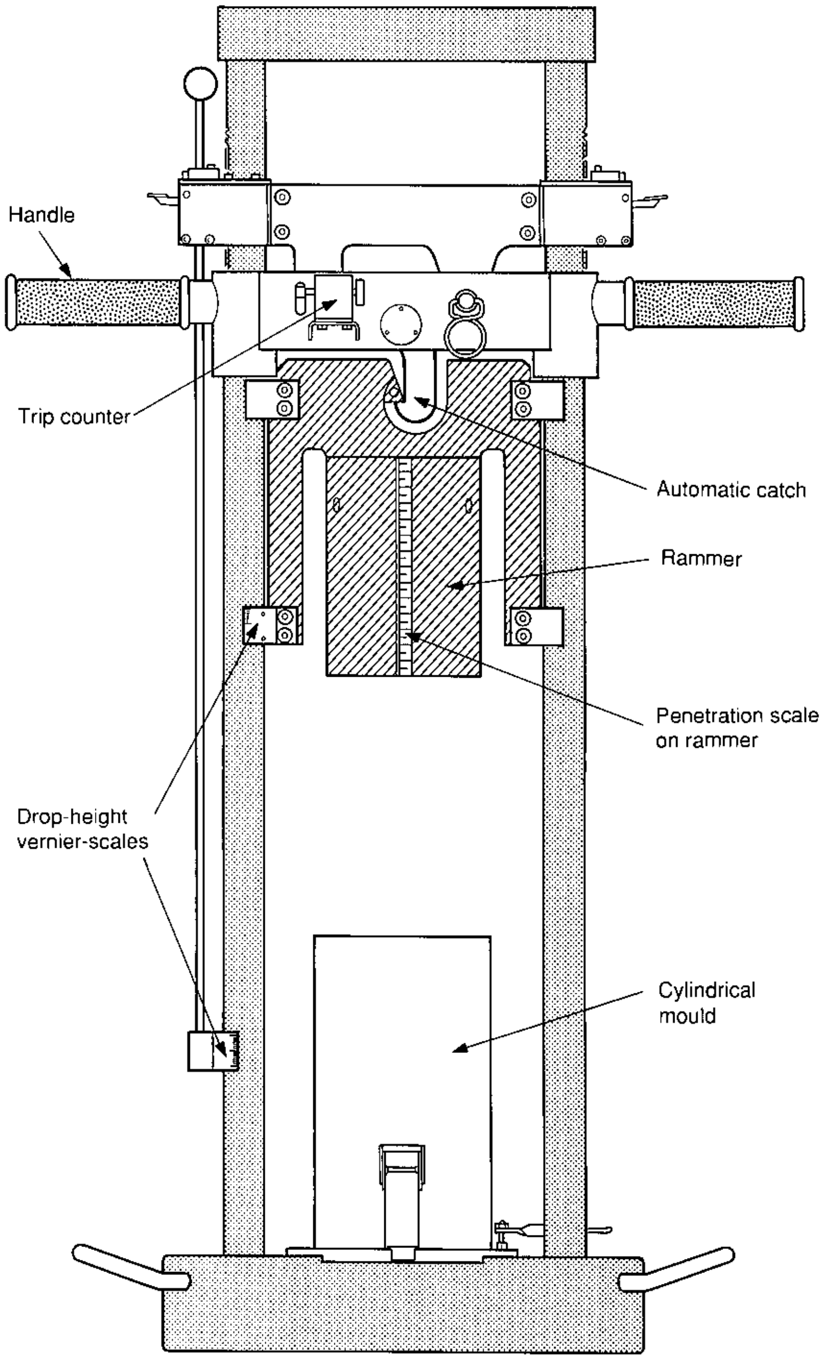


Fig.11.7 The apparatus for the Moisture Condition Test.

Soil: Heavy clay Moisture content: 26.3 per cent

Number of blows of rammer 'n'	Penetration of rammer into mould (mm)	Change in penetration between 'n' and '4n' blows of rammer (mm)
1	41	33.5
2	57.5	33
3	67	33.5
4	74.5	26.5
6	84	17
8	90.5	10.5
12	100.5	0.5
16	101	
24	101	
32	101	
48	101	

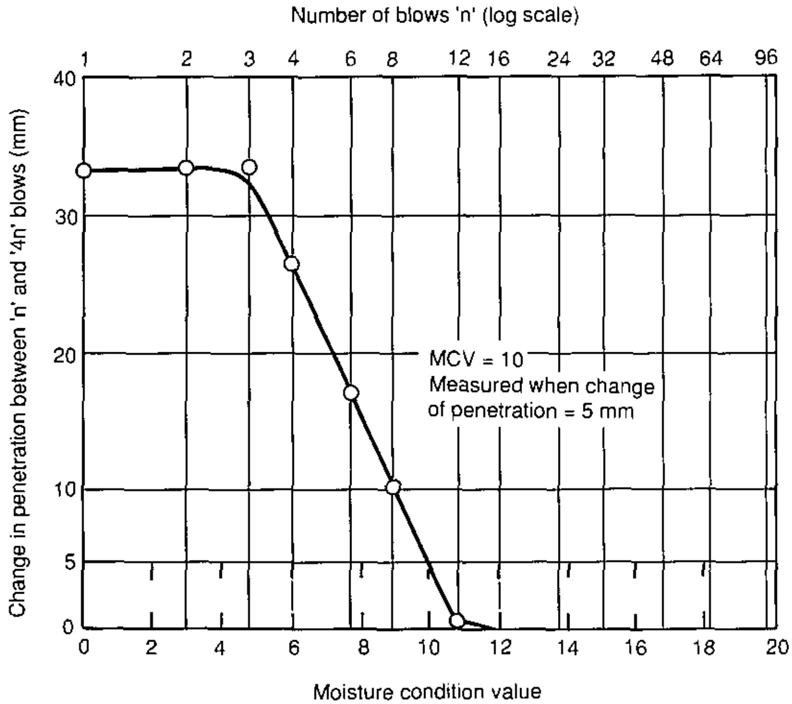


Fig.11.8 Results of field tests to determine the Moisture Condition Value (MCV) for a heavy clay.

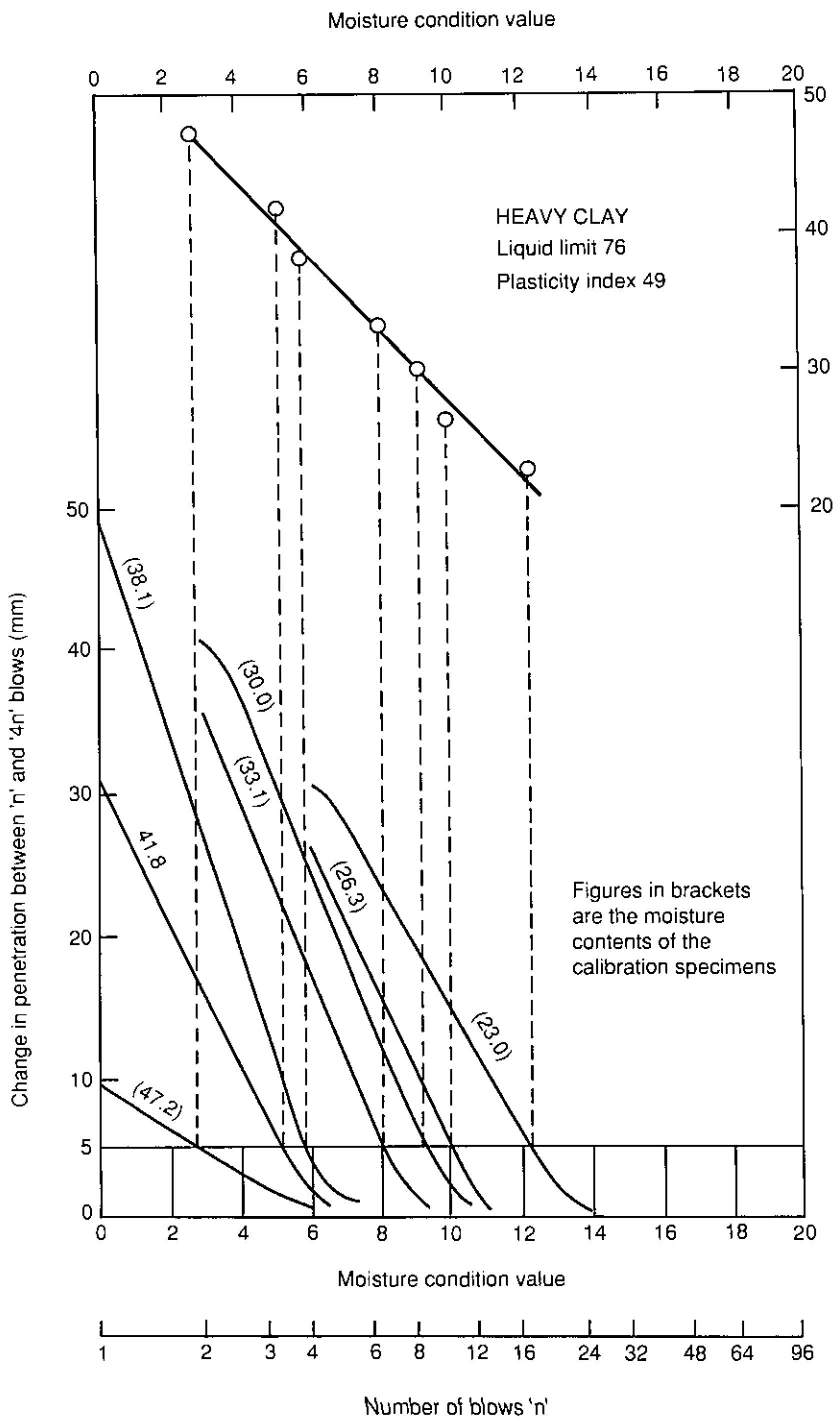


Fig.11.9 *Calibration chart for the determination of moisture content from the Moisture Condition value of a sandy clay.*

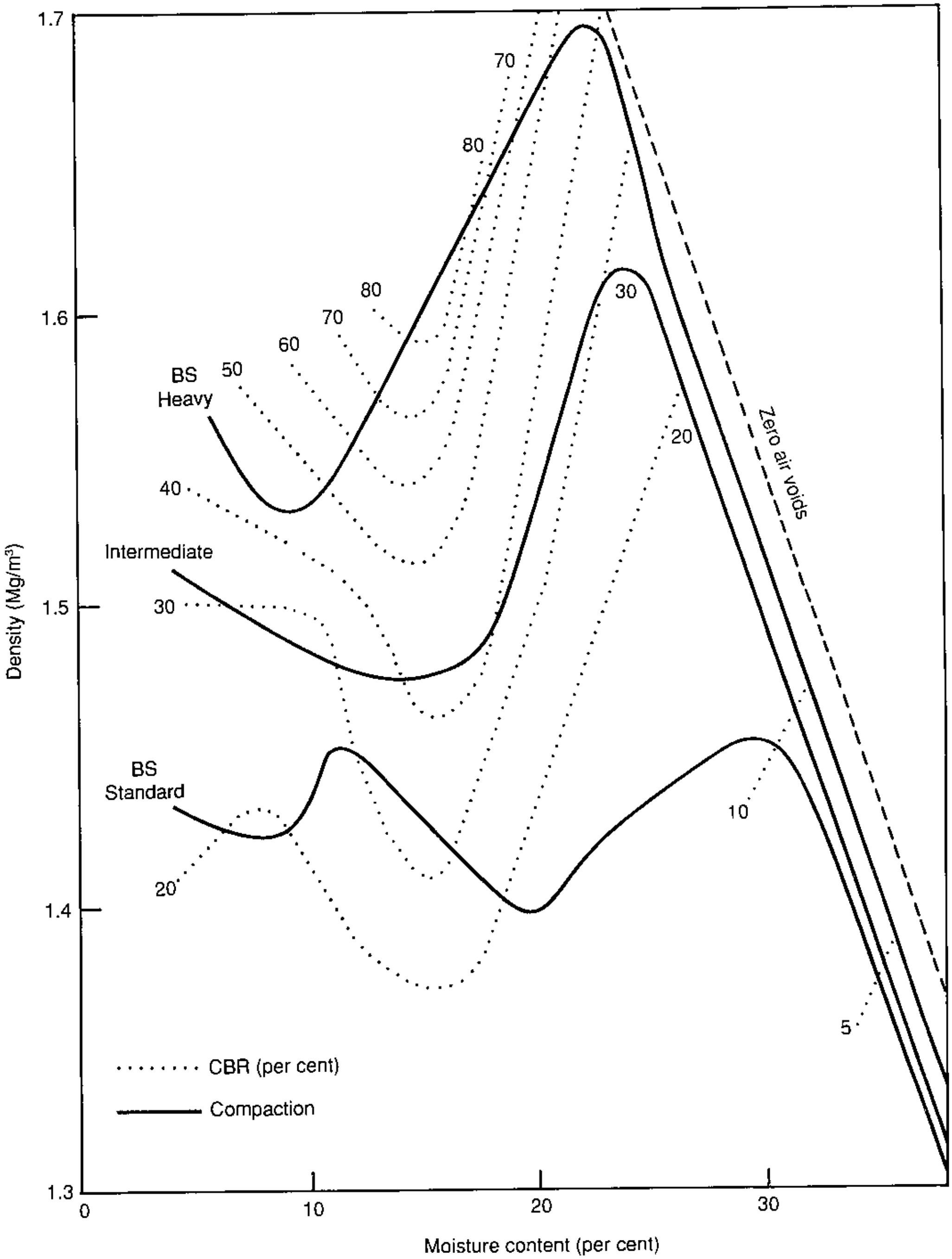


Fig.11.10 Compaction characteristics of an expansive silty clay from Sudan. (LL = 64-76, PL = 38-42.)

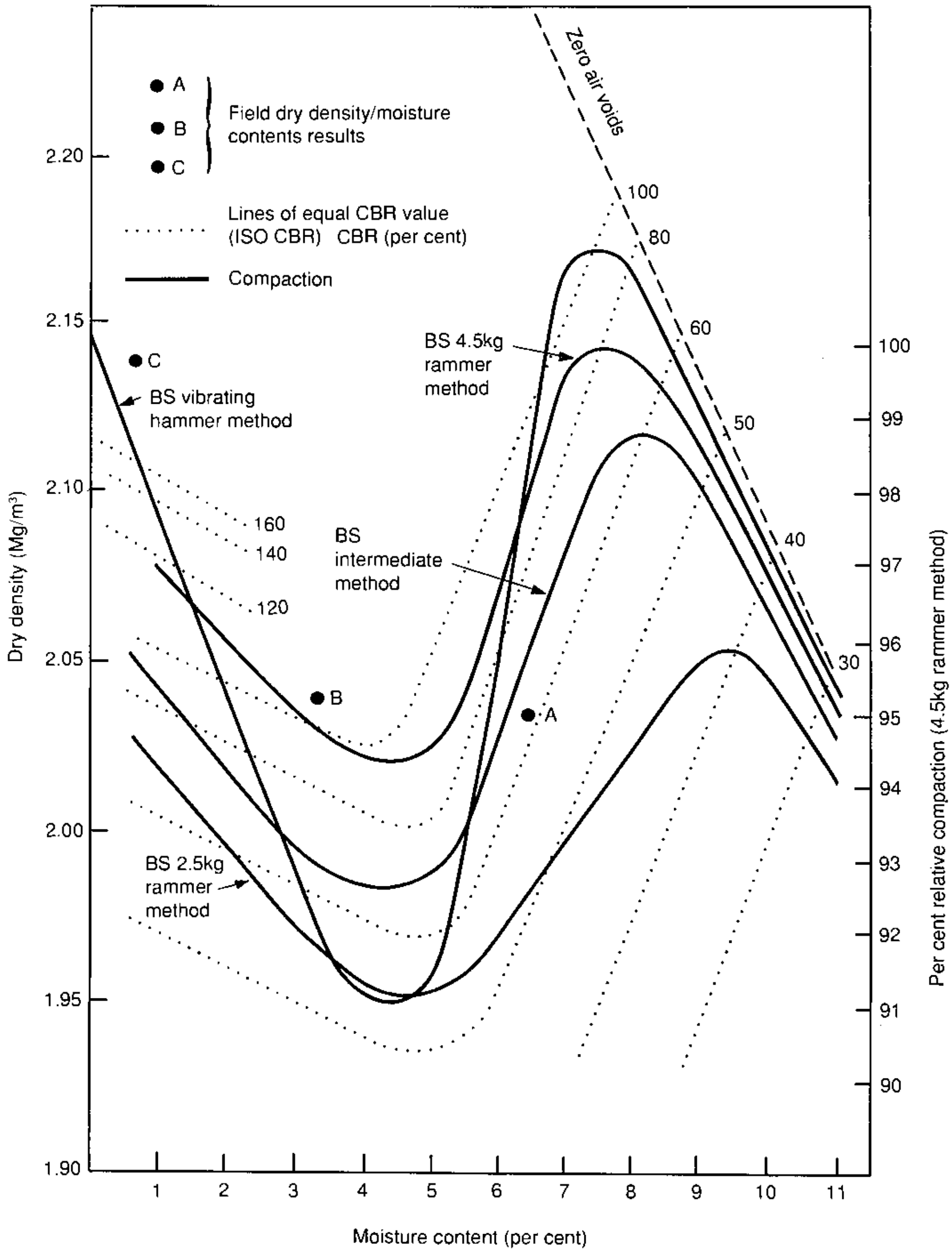


Fig.11.11 Compaction characteristics of a clayey gravel from Kenya (LL = 33, PL = 15-22).

provides a means of compacting the clayey gravels which are used in many tropical countries for road bases and surfacings. The CBR values obtained on the dry-compacted soil are of the same order as those obtained on soil compacted and tested at the optimum moisture content. This is an unexpected result since one would expect clayey gravels compacted to constant density to become stronger at lower moisture contents. Indeed, this is what happens when a soil compacted at optimum moisture content is dried to a lower moisture content. As the moisture content decreases, higher suction forces develop in the clay fraction and CBR values increase to 3 or 4 times the value at optimum moisture content. Why is it that the same high values are not found in the dry-compacted specimens?

The answer has interesting practical applications. During dry compaction the bonds in the soil created by the high suction forces are at least partially broken down; the clay then functions as a non-cohesive dust. Much of it will blow away if the gravels are left exposed to traffic and weather. If the compacted soil is subsequently wetted, cohesion will develop in the clay fraction and will increase as the soils dry out thereafter. Thus the soil develops both strength and stiffness. This is the reason for the very good performance of these clayey gravels as road bases which has sometimes puzzled road engineers from more temperate climates (see Grace and Toll (1987) also Queiroz *et al* (1991)). When densely compacted, these clayey gravels are of low permeability and, under the protection of a bituminous surfacing, they are not weakened by wetting up in subsequent rainy periods. Indeed, when exposed by inadequacies in the surfacing, they retain their natural cohesion and are less prone to break into potholes than are bases of crushed stone when so exposed.

11.3 Cuttings and slopes

11.3.1 General considerations

Cuttings through unweathered rock can often stand at a near vertical slope. More usually, slopes and cutting faces consist of weathered rock or of soil, sometimes in a precarious and unstable condition. Many mountain ranges are of quite recent geological origin and the rock slopes are deeply weathered with the weathered material on the move, evolving towards a more stable topography. On steep natural slopes the instability is usually evident to the eye in recent landslips. Lower down the soil catena, instabilities may not be so obvious but there can be slip planes and zones of impermeable soil below the ground surface deriving from the parent geological formations. In all areas, the stripping of natural vegetation to bring land near the road into cultivation will increase both the speed and the amount of surface water run off, eroding the valuable top soil and intensifying problems of slope stability.

These instabilities always derive from an accumulation of water in the soil. Most frequently, slips occur during heavy rain when the accumulated water in surface layers both reduces the natural cohesion of the soil and increases its mass. Deeper slip failures may come later as the zone of increased moisture content moves down through the soil to accumulate over any less permeable layers below. In the design of roads it is necessary to provide defence both against the erosive forces of surface water (including the rain itself) and against instabilities caused by subterranean water movements. These usually need separate consideration but there is a general principle which must always be followed. The construction of the road should encourage the rapid and safe movement of water from the area above the road to the area below it. In no circumstances should the construction produce a barrier to the gravitational movement of surface water or ground water.

Control of surface water erosion in land above the road does not normally lie within the responsibility of the road engineer, but where the land is being cleared for farming, he will need to collaborate with the agriculturalists both to minimise the surface erosion and to work out the maximum rate of run-off that his surface water drainage system will have to accommodate. Below the road he has the responsibility to make sure that the accumulated surface water from the surface water drainage system is discharged safely into natural water courses.

11.3.2 Slope stability

Methods of analysis involve measurements of the influence of moisture contents and state of compaction on the strength of the soil together with an estimation of the stresses likely to be developed in the soil in situ. The purpose is to indicate a safe face angle for the slopes. Usually, a safety factor of 1.5 is used to bring the calculated stresses in the soil below those which the tests have revealed the soil can carry. The calculation of stresses involves the classic slip circle analysis (see Bishop (1955)). Slip circle analysis presumes a uniformity in soil type. Actual failures do sometimes assume the typical slip circle form, but a uniform soil profile is somewhat rare, particularly in residual soils, and it is more usual for slips to occur along planes of weakness in the vertical profile. This does not impugn the value of slope stability analysis. Such analysis may be a necessary component in the survey work that is necessary to determine the slopes at which soil cuttings should be formed. It is particularly important in investigating the causes of slope failures and in determining what remedial works are necessary.

11.3.3 Surveys for slope stability

An integral component of any survey must lie in the assembly of data to catalogue the performance of natural and man-made slopes in the soils encountered along the length of the road. This catalogue should include both failed and stable slopes and, with the failed slopes, the forms of failure should be identified so that any special measures of protection can be included in the design. These procedures have been formalised in a hydrological design manual for slope stability in the tropics by Anderson and Kemp (1989) summarised by Anderson and Lloyd (1991).

In discussing the work of soil survey we always seem to have in mind the needs of a surveyor working in a country previously totally unknown to him and his associates. But this is rarely the case. These days, engineers, and others concerned with soil attributes, have worked in most parts of the world and the benefits of this experience ought to be available, both to practising engineers and to students. In the older developed countries it is available in both the experience of local engineers and in publications, but over most of the world's surface it is not readily available. Herein lies the value of the landscape classification techniques described in Chapter 2.

Aerial photographs can be used to identify existing slopes and to indicate, from changes in

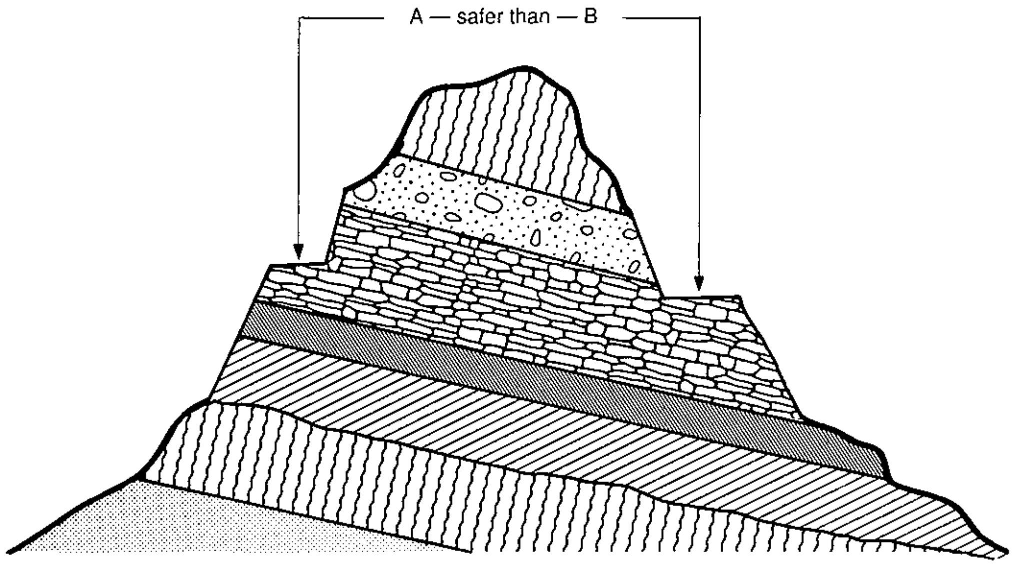


Fig.11.12 Road location in relation to inclination of rock strata.

vegetation and ground contour, areas of potential slips. In hilly country it may be possible to use photographs taken from the ground to obtain indications of areas of instability above and below the road line (Heath and Dowling (1980)). Seismic refraction and other geophysical techniques can be useful in determining sub-surface ground conditions (see Stewart and Celis (1977)).

Where well defined strata appear in the parent rock it is an advantage to locate roads over ground where the strata dip downwards into the hill and, by the same token, to avoid putting roads across hillsides where the strata are inclined in the same direction as the ground surface (see Fig. 11.12). All water courses crossing the road line should be identified during the survey and it is important to establish their susceptibility to erosion. Above the road it may be necessary to provide defence against water-borne boulders. Below the road, where the water courses will take the increased flow from surface water draining off the road, protection will also be necessary. In the area immediately below the road it may be necessary to construct water chutes and stilling chambers to control erosion.

11.3.4 Design and construction

The inclination of cuttings will have been decided at the survey stage. One decision to be made is whether the slopes should be continuous or whether they should be benched. Benching originated in the days of hand construction, the height of a bench being limited by a man's reach. On deep cuts it may still be a useful construction expedient, enabling excavating plant to cut and shape the excavation in well defined stages (see Fig. 11.13) and simplifying the maintenance of the cutting faces. The slope of the inclined faces of the cutting cannot be increased when benching

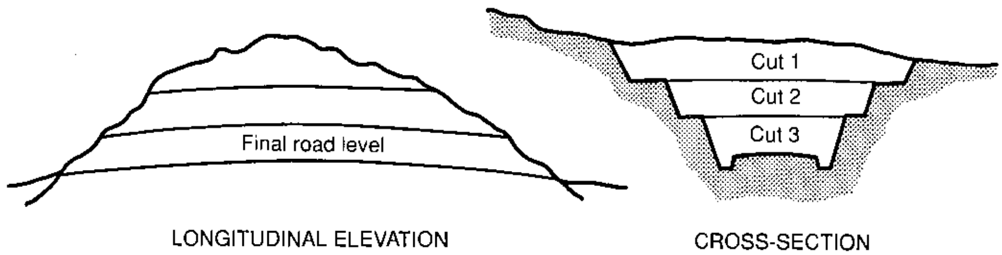


Fig.11.13 Excavation of benched cuttings.

is used and more excavation is therefore needed than when the side slopes are continuous. It may be that benching has advantages mainly when making cuttings through sound or partially decomposed rock which will stand at an angle not far from the vertical. Opinions vary on the direction in which the tops of benches should be inclined. If this inclination is down towards the face of the cutting, surface water run-off will discharge on to the cutting face thereby possibly causing surface erosion. Downwards into the cutting itself will produce concentrations of run-off water at the back of the bench which may produce instability in the cutting. Unless the soil in the cutting is particularly susceptible to erosion it is better to slope the benches down towards the face of the cutting. If they are sloped the other way, a paved drain should be provided to remove surface water (see Fig. 11.14).

Similar considerations apply to cut-off drains. These are sometimes provided in the slope above a cutting to collect and remove surface water run-off from above. These are also sources of weakness unless they are lined to prevent the water from penetrating into the slope.

Drainage of surface water run-off and ground water may be combined. This has been done in Zimbabwe on the plains of decomposed granite where there are likely to be perched water tables over a zone of clay accumulation situated between one and two metres below the ground surface. Side ditches are dug about two metres below the formation level of the road to intercept this clay zone at the ends of cuttings.

On tortuous alignments, such as are often necessary on steep hillsides, the pavement width is increased at bends to facilitate turning traffic. Blind bends are a double hazard since it is at these places that ground slips are particularly common. Normally a paved drain will be provided between cutting faces and the road. On such hill roads it may be desirable to cover the drain with concrete slabs so that the drains do not readily become blocked by falling debris. It may even be possible to dispense with the drain altogether by carrying the bituminous surfacing right up to the face of the cutting so that its inside edge also serves as the storm water drain (see Fig. 11.15). An extra width of pavement can thereby be obtained, both improving access on the bend and providing a platform from which fallen debris can be removed without difficulty.

In mountainous country, roads frequently have to zig-zag up the mountain face to provide a negotiable gradient for the road. When first built, these roads are often laid out by eye and

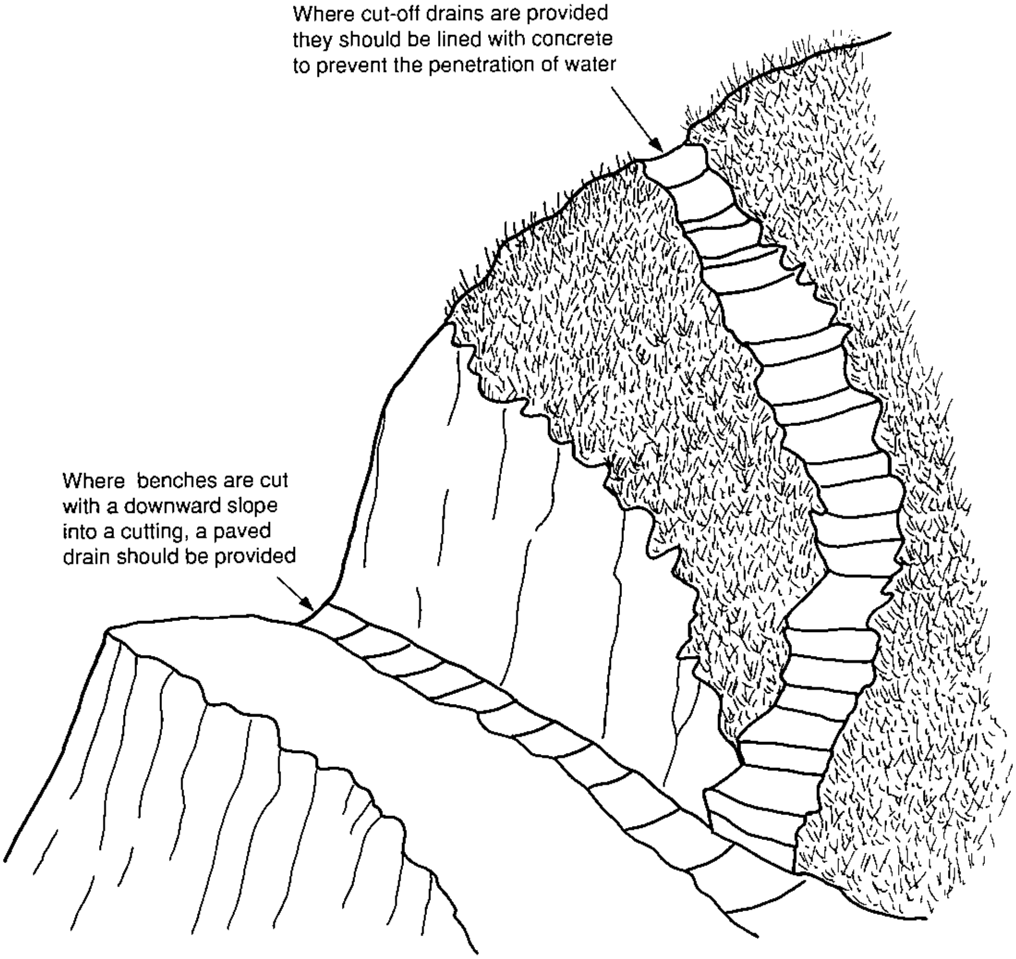


Fig.11.14 Drainage of cutting faces.

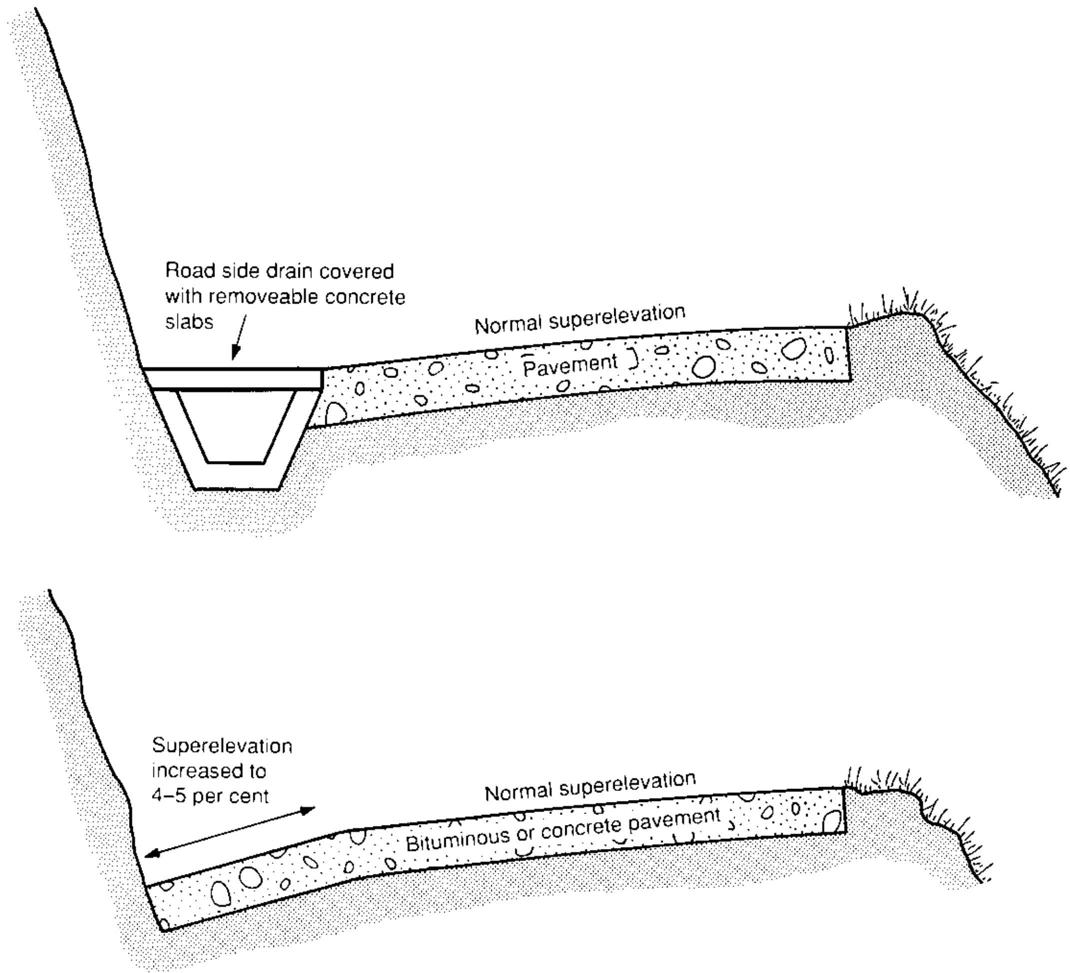


Fig.11.15 Surface water drainage at bends on hill roads.

constructed with rudimentary equipment. Material excavated from the hillside is used to build the road platform, and instability can result if this material is not well compacted, keyed in place and adequately drained. Masonry walls are often needed to support the filling (see Fig. 11.16). As the walls are built, workmen can operate behind them to compact the filling in place.

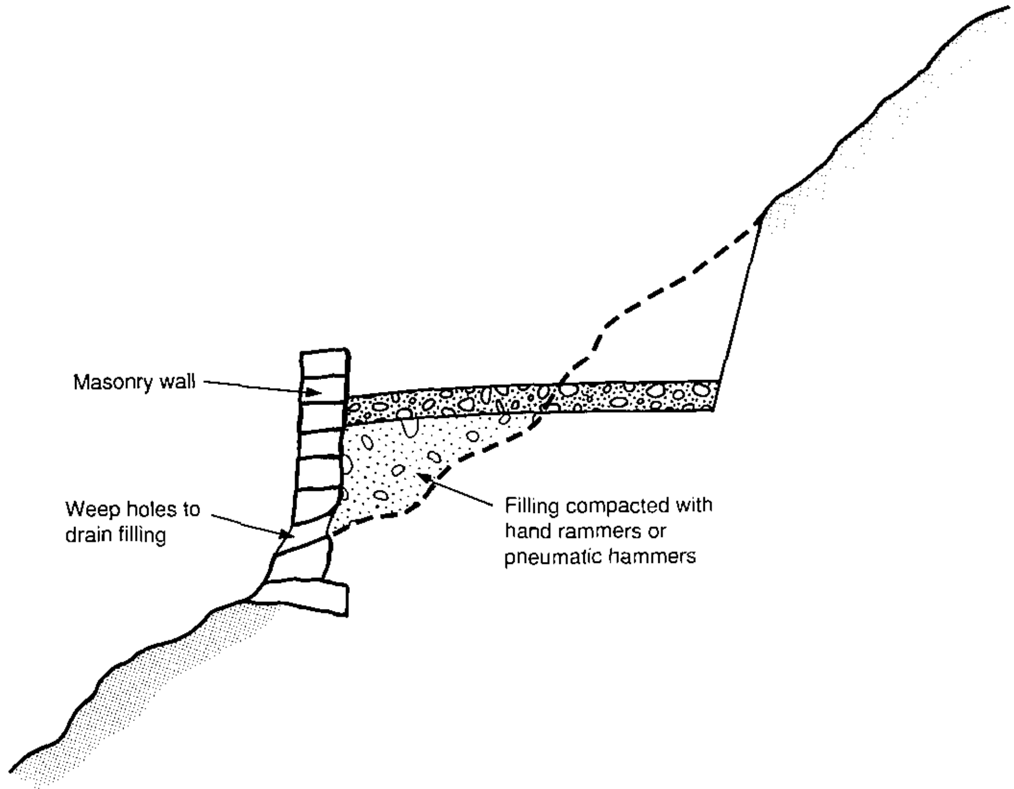


Fig.11.16 Masonry retaining wall.

11.4 Construction over compressible soils

11.4.1 Types of compressible soil

There are broadly two groups of highly compressible soil. One, widespread in the tropics, consists of the silty clays and clayey silts occupying river deltas and coastal areas. These are transported soils that have been deposited under water. The other is peat, an organic soil that has developed in situ from decaying vegetation. In temperate regions these two soils frequently occur together, the peat deposits having developed and sometimes been covered by more transported soils as river levels have changed. In temperate and cold climates, peat, deriving from the growth of mosses, can build up to a thickness of 1.5 metres or more above water level. But in the tropics, organic materials above the water table soon decays, and persistent organic soils are found only in marshy areas where, below water level, the decay of vegetable matter proceeds only very slowly e.g. the Sud of the Southern Sudan. The poorly consolidated silt-clays of river deltas in the tropics rarely contain organic soil except, possibly, in thin deposits on the surface associated with present-day lagoons. In them, organic material and silty-clays are usually intimately mixed.

The distinction between the two groups is important because of their different consolidation characteristics. Both exhibit primary consolidation i.e. the squeezing out of water when the soil is loaded. This process is usually completed after two or three years and, with silt-clays, it can be accelerated by the movement of water through naturally occurring lenses of sandy soil or through man-made drainage paths in the soil. With organic soils the decay of the vegetable matter produces secondary consolidation which can continue for a long time after primary consolidation is completed.

When unconsolidated soils occur in a wide flood plain they are frequently of fairly constant thickness. In Thailand, for instance, they are about 12 metres thick in the region 50 km north of Bangkok, increasing to 15 metres in thickness in the vicinity of the capital. They can vary considerably in thickness in areas where the underlying strata have been deeply dissected by river systems. They are usually underlain by firmer sandy soils laid down at a time when the rivers flowed faster. They increase in density with depth, having been consolidated by the weight of soil above. It can normally be assumed that with thicknesses over 20 metres the soil below this depth has been consolidated to a state at which it will not be affected by the loading of road embankments. Beds of peaty soil are not normally more than 3 metres in thickness but can be thicker in sluggish rivers that have, at an earlier time, cut a deep channel through the surrounding ground.

11.4.2 Construction over unconsolidated silt-clays

On all but very lightly trafficked roads, embankments are necessary to provide a firm support for the pavement and the traffic that it will carry. They may also be needed to raise the road above flood level. There are two primary considerations in the design of such embankments. One is to determine the amount of settlement that will occur in the soil under the load of the embankment. The other concerns the rate of loading, which must be controlled below a critical level at which pore water pressures can cause slip failures to occur below the toe of the embankment. Useful reviews of these considerations have been published by Lewis *et al* (1975) and in the proceedings of a symposium held at the Transport and Road Research Laboratory (1976).

Consolidation tests undertaken on laboratory samples can be used to examine the performance of soils under load, (see Section 2.15). Such testing usually provides on accurate indication of the amount of settlement likely to occur under an embankment but the associated theory usually grossly overestimates the time required for the settlement to occur. This is because most deposits of unconsolidated silt-clays contain horizontal lenses of sandy soils deposited during periods of heavy flooding. These lenses are permeable and provide paths for the water to escape from the loaded soil. The presence and effect of such layers can be detected by in situ testing involving the use of piezometers at different depths in the soil. This is done by applying a small excess pore water pressure in the vicinity of the piezometers and monitoring the rate of flow through the piezometers for 24 hours or more. As can be seen from Fig. 2.15, such in situ testing often provides a reliable method of estimating the time required for consolidation of soils under the load of an embankment. This time is usually four or five times less than the time estimated from consolidation tests on small samples of the soil in the laboratory.

Vertical drainage can be used to accelerate the rate of consolidation. Originally, such drainage consisted of columns of sand, about 500 millimetres in diameter, installed at regular intervals over the area below the embankment. Nowadays wick drains are employed. These are made from cardboard or other fibrous material which can be easily inserted into the soil using a removable hollow metal tube. Vertical drainage came into use as a result of the long settlement times estimated from laboratory consolidation tests and before it was realised that many unconsolidated soils contain natural horizontal drainage paths.

One method of reducing instability problems is to employ lightweight filling for the embankment. Fly ash, the waste material from coal-fired electricity generating stations, may be available in some areas. Sawdust and rice husks have been used in circumstances where the need for stability is of prime consideration and where future settlement within the embankment as the material decays is judged not to be important. They may, for example, be used on more lightly trafficked roads to help in constructing a shallow working platform for construction plant.

On such shallow embankments there are advantages in using pavements of concrete; its rigidity helps in reducing the effects of uneven settlement. Joints should always be dowelled to transfer traffic loading between slabs. There are examples of concrete pavements over organic soils and over unconsolidated silt clays that, despite settlement of 200 millimetres or more in the foundations, have continued to present an even running surface to traffic for many years. There is, of course, the problem that the level of culverts for cross drainage must be preserved by providing foundations that will not settle e.g. by piled foundations through to firm strata, but, on lightly trafficked roads at least, the resultant bump in the road profile is of little consequence and its effects can be minimised by carrying one end of the adjacent road slabs onto the culvert superstructure as shown in Fig. 11.17.

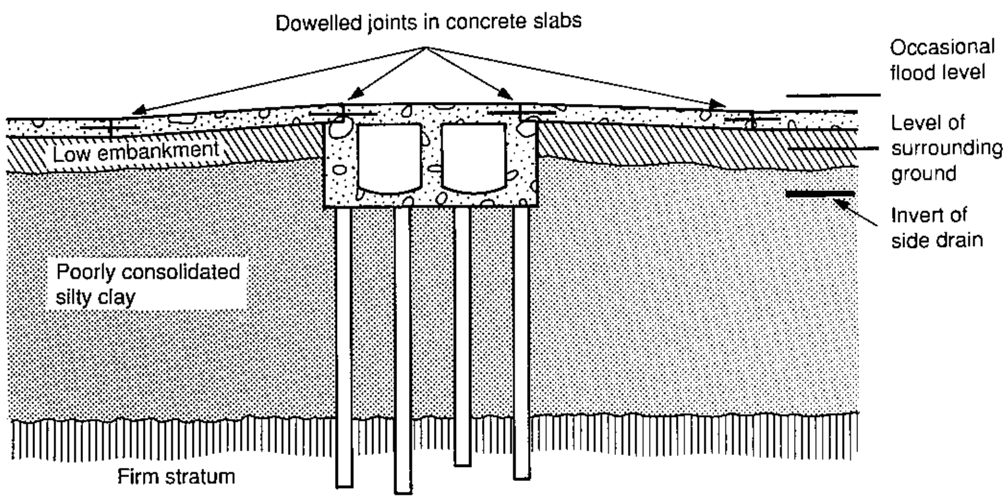


Fig.11.17 Settlement of concrete road over compressible soil adjacent to culverts.

Slip circle failures of the classic form described by Bishop (1955) are likely to occur under the toe of the embankment if the slopes of the embankment are too steep and if the rate of application of load is too rapid for pore water pressures to dissipate. Piezometers and inclinometers can be installed in the soil to detect high pore water pressures and soil movements. Control charts can be prepared for particular embankments by which engineers can employ a factor of safety in monitoring the pore water pressure and controlling the rate of embankment construction to keep this pressure below a critical level. An example of such a chart is shown in Fig. 11.18 (see also Margason and Symons (1969)). One sign of incipient failure is in the tilting of bushes and small trees away from the road immediately beyond the toe of the embankment. This signals an immediate need to delay further embankment construction and to consider reducing the slope of the embankment faces.

11.4.3 Construction over organic soils

It is always best to remove and replace peaty soils of high organic content because of the great difficulty in consolidating them to a level that further settlement will not occur. Furthermore, even when consolidated, they provide a very weak foundation for roads. Wherever possible, roads should be located so that they avoid areas of peaty soil. There are two typical circumstances when this may not be possible. One lies in traversing large areas that are perpetually flooded e.g. mangrove swamps, and the other is in the crossing of turbid rivers. In both cases there is usually some movement of water, however slow, and it is necessary either to locate the road embankment so that it does not impede this flow or to provide for cross-drainage. Culverts are useless in such circumstances since they soon become choked with vegetable debris. The only answer with turbid rivers and streams is to provide bridges founded on firm strata below.

Where soil suitable for embankment construction is readily available, lagoons and swamps can be traversed using the same methods as are used for construction over unconsolidated silt-clays. A good example lies in the construction of the road over the Caroni swamp in Trinidad, reported by Osborne (1960). Since that time there have been developments in geofabrics which, laid on the soil surface below the embankment, can increase resistance to lateral displacement.

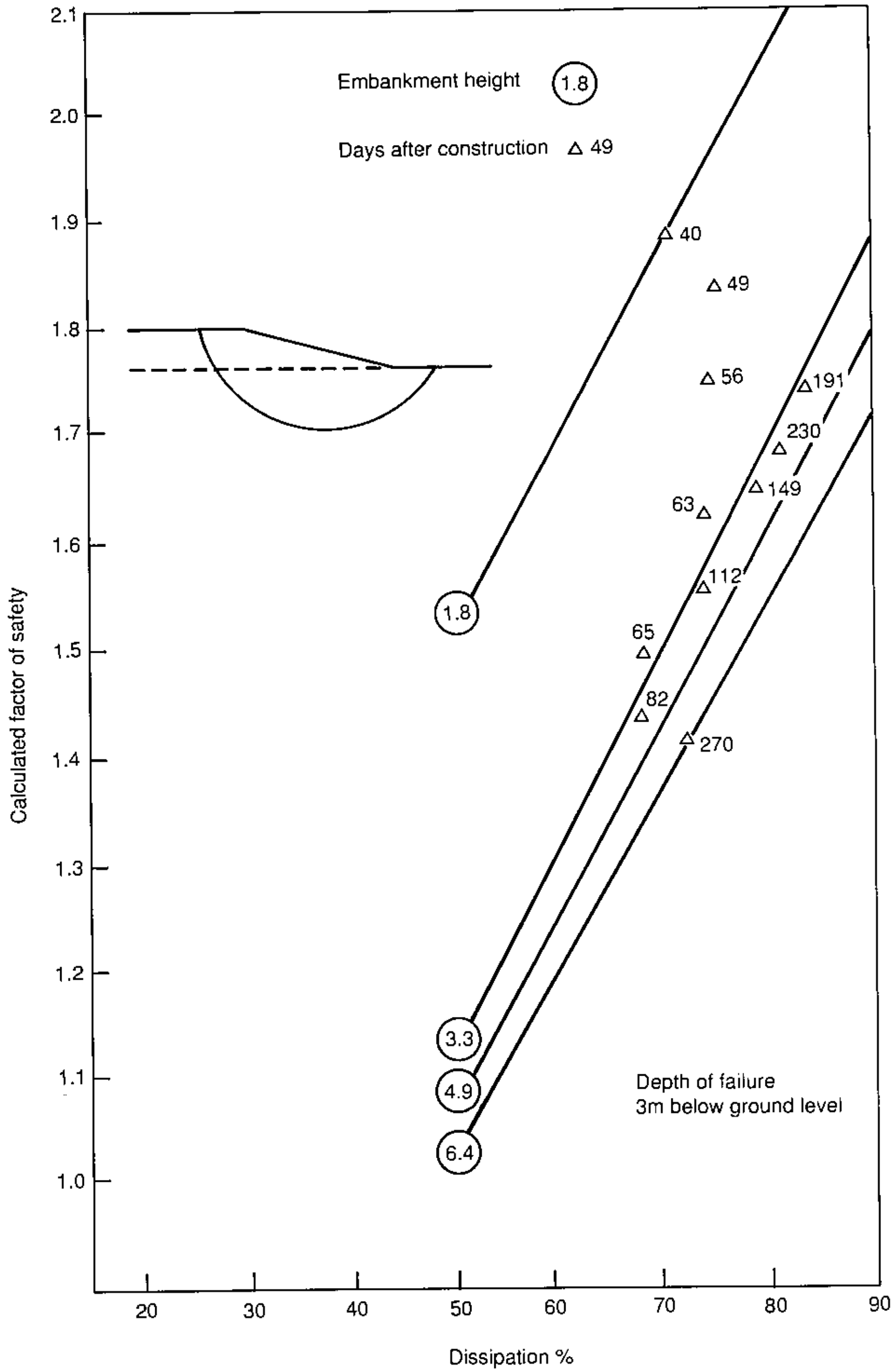


Fig.11.18 Stability chart for circular failures.

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12 Earth and gravel roads

12.1 Introduction

In most countries outside Western Europe substantial proportions of the road networks are surfaced with earth or gravel. Earth roads consist of tracks made from the in situ soil, normally suitable for light traffic in dry weather. The provision of a gravel surface is usually the first stage in making an all-weather road designed to particular standards of alignment and traffic carrying capacity. There can be no doubt about the weaknesses of earth and gravel roads. In dry weather, traffic generates clouds of enveloping dust. Unless the road surfaces are assiduously maintained, their surfaces become corrugated and potholed and, in wet weather, they can become foundrous. Vehicle operating costs are higher than on roads with permanent surfacings. Investigations in Africa and Brazil have shown that this increase in costs may be quite small on well maintained gravel roads, but in many countries it is proving impossible to provide the regular treatment needed to prevent the surfaces from deteriorating under the action of traffic and weather. Under these circumstances vehicle operating costs can rise by more than 50 per cent (see Watanatada *et al* (1987) and Parsley and Robinson (1982)). Road accidents also occur more frequently. In Kenya, Jacobs and Sayer (1976) report that 18 per cent of all accidents occurred on gravel roads which carried only 5 per cent of the total traffic. There are thus powerful incentives in all countries for increasing the proportion of roads with bituminous or concrete surfacings and methods of determining the economic justification for such works are now well established.

In many countries the minor roads are likely to remain surfaced with earth or gravel for some years and new gravel roads will also be needed, particularly in rural development projects. The major problem lies in the funding and the execution of effective maintenance for the existing networks of earth and gravel roads. Here, tropical countries have two advantages. One is that their climates are usually less inimical to the performance of gravel roads than in those regions of the world where damp, cold and frosty conditions prevail. The other lies in the relative abundance in many areas of good road making gravels deriving from the weathering of surface rocks.

12.2 Design and construction

Gravel roads are frequently developed along the line of tracks that have been beaten out by travellers on foot. Often, these travellers will have found a convenient route, but it is always wise to consider whether some relocation is desirable to produce grade lines appropriate for motor traffic. Steep slopes should be avoided. On grades of more than 6 per cent there is likely to be severe erosion of gravel and earth surfaces in heavy rain. It may be desirable to provide pavements of concrete or bituminous surfacings where steep gradients are unavoidable. Sometimes there will be two tracks, a dry season alignment following valley bottoms connecting centres of human habitation, and a wet season alignment at a higher level avoiding areas that are subject to flooding. For a simple motorable road it will usually be best to follow a high alignment to preserve access in wet weather. A ground hugging alignment should be followed, keeping as far as possible to level contours.

A new road constitutes an interference with the natural drainage pattern of the area and care is necessary to avoid setting up concentrations of water flow that can cause erosion of the surrounding area. Roadside drains will be necessary with turn-outs sufficiently closely spaced to disperse the water without damage. Cross drainage will be necessary where the road traverses sidelong ground. Simple small culverts can be built by hand but it is an advantage to have access to precast concrete pipes of 350-500 millimetres diameter. Where such roads are being built by an outside agency, it is helpful to arrange for a supply of such pipes to be left behind for use by road maintenance teams at places where cross drainage proves to be inadequate.

In heavily forested areas it is commonly observed that roads running in an east-west direction perform better than roads running north-south. This is for the somewhat obvious reason that they are exposed to the drying effects of the sun for a greater part of the day. There are advantages in cutting back tall vegetation where this puts the road in shadow. Earth and gravel roads require steeper cambers than bituminous surfacings for rainwater to be shed satisfactorily to the side of the road. Cambers should be between 1 in 25 and 1 in 15 (4-7 per cent) with the steeper cambers used on steeper alignments to reduce the longitudinal gulleying commonly found on such alignments.

Some enthusiastic engineers ask how the principles of pavement design can be used to determine an appropriate thickness for the gravel running surface. Gravel road surfaces are a wasting resource. Under the action of traffic and weather, the finer particles disappear as dust and the coarser particles are displaced into the side ditches and beyond. Some can be brought back on to the road surface during maintenance by grading in the wet season but the overall losses are considerable. They increase with increasing traffic intensity and are also influenced by the nature of the gravel and by prevailing weather conditions. Annual losses of dust up to 25 tonnes per kilometre on roads carrying 100 vehicles per day have been reported by Jones (1984a) in Kenya, and with the coarser material lost at the sides of the road there are reports of total annual losses between 70 and 300 tonnes per kilometre (see, for example, Jones (1984b)). Precise calculations of desirable pavement thickness are generally only of academic interest. It is usual to provide gravel to a nominal compacted thickness of 150-200 millimetres with the anticipation that a further similar application will be needed in the next 2-10 years, depending on traffic intensity. An exception to this general principle is in areas where subgrades are weak and traffic is substantial. In these circumstances some measure of subgrade protection is necessary to prevent the rapid formation of deep ruts in the wheel tracks and the closing of the road. Various authors have developed empirical thickness design methods usually based on subgrade CBR values. An example based on extensive work by the United States Army is given by Ahlvin and Hammitt (1975).

Gravel losses by the action of traffic and weather are considerable. In parts of Kenya, Malawi and elsewhere there are indications that resources of gravel near public roads are being exhausted and supplies of good gravel are being reserved for the construction of bituminous-surfaced roads.

In many countries there are rural communities not yet served by motorable roads. Inevitably it is one of the greatest ambitions of the local people to have such a road to provide ready access to markets and to schools and medical services. They are often willing to provide the labour necessary to build rudimentary roads. But they lack the skills and resources to do so on their own and some external intervention is needed to provide these resources. Local self-help schemes for road building now feature in many rural development projects and indeed they are a key element in reducing migration to the towns (see Beenhakker *et al* (1987), Edmonds and de Veen (1981), Morris (1989), Thomas and Hook (1977) and Transport and Road Research Laboratory (1979)).

Such rural roads will survive only if an organisation is put in place to keep them adequately maintained. In this we can see an important feature of emerging local government through which the necessary taxation revenues and resources can be channelled.

12.3 Properties and types of gravel

Typical particle size distributions for gravel surfacings taken from Overseas Road Note No 2 (TRRL, (1987)) are given in Table 12.1.

TABLE 12.1
PARTICLE SIZE DISTRIBUTION FOR GRAVEL SURFACINGS

BS test sieve (mm)	Percentage by mass of total aggregate passing test sieve		
Nominal max size	*20mm*	*10mm*	*5mm*
37.5	100	-	-
20	80-100	100	-
10	55-80	80-100	100
5	40-60	60-85	80-100
2.36	30-50	45-70	50-80
0.425	15-30	25-45	25-45
0.075	5-15	10-25	10-25

(*) Not less than 10% should be retained between each pair of successive sieves specified for use, excepting the largest pair.

TABLE 12.2
PREFERRED PLASTICITY CHARACTERISTICS FOR GRAVEL SURFACINGS

Climate	*Liquid Limit not to exceed* (%)*	*Plasticity Index range* (%)*	*Linear Shrinkage (%)*
Moist tropical and wet tropical	35	4-9	2-5
Seasonal wet tropical	45	6-20	3-10
Arid and semi-arid	55	15-30	8-15

(*) Higher limits may be acceptable for some laterites or concretionary gravels that have a structure that is not easily broken down by traffic. Lower limits may be appropriate for some other gravels that are easily broken down by traffic. Any variation from these limits should be based on local experience.

Texturally these represent clayey gravels or clayey sands and it is desirable for the plasticity of the fines to be varied according to the prevailing local climate with materials of lower plasticity being used in wetter climates as indicated in Table 12.2.

These Tables give guidance and can be used to provide targets for contractual specifications in selecting between alternative deposits and in controlling the quality of material extracted. The materials in deposits should be bulldozed into stockpiles to minimise the effects of any striations in the deposits and care is often necessary to avoid penetrating into more plastic soils which usually underlie the deposits.

The characteristics and testing of the different types of gravel likely to be available have been reviewed in Chapters 2 and 3. Each has some idiosyncrasies relevant to its use as a road surfacing. Lateritic gravels are often ideal for the purpose. In some areas the gravel particles may be only partly laterised and, in addition to particle size analysis and plasticity tests, the hardness of the particles must be evaluated, rejecting material from deposits with nodules that are too weak. Quartzitic gravels and colluvial gravels may contain large particles. It is desirable to restrict the maximum size of aggregate to less than 37.5 millimetres. Particles of larger size hamper the blading operations necessary to secure and maintain an even surface. Gravels from decomposing basic rocks have been successfully used. These too may contain large particles. Such particles can often be broken down to a convenient size by the use of a grid roller during compaction. The minerals in these larger particles are usually partially weathered and it is a matter for local experience to determine whether they will retain sufficient mechanical strength to serve for the few years before regravelling becomes necessary. Volcanic scoria are usually mechanically rather weak but their fines are of low plasticity. They tend to produce rather dusty surfaces and regravelling may be necessary at shorter intervals than with other types of gravel. Alluvial gravels taken from near existing rivers are usually single-sized. This and their rounded particle shape make them unsuitable as surfacings, but in deltaic areas associated with ancient river systems, deposits of terrace gravels containing sand-silt-clay fines can often be found which may be suitable for use.

One of the uses of the landscape classification systems described in Chapter 2 lies in characterising the modes of occurrence of the different types of gravel available in a particular country and in building up knowledge of how they perform under traffic. Gravels from different sources vary considerably in their propensity to become corrugated and potholed under traffic and in the rate of gravel loss under the action of traffic and weather. The development of effective road maintenance strategies require a knowledge of these characteristics.

12.4 Corrugations

Corrugated road surfaces are a familiar source of frustration to travellers. The reason why they form in such a regular pattern is a frequent and unresolved subject for debate. Theories abound but, as yet, no satisfactory explanation has been produced and there is no method of determining the propensity of gravels to become corrugated except by observation of their performance in use.

A typical corrugated road surface is illustrated in Fig. 12.1. The pitch of the undulations is constant on any one site, usually of the order of one metre, but on different sites and with different materials it can vary between 0.5 and 1.5 metres. The amplitude i.e. the height of the crests above the hollows, can be 50 millimetres or more on a badly maintained road. The material in the corrugations is extremely well compacted and hard. It cannot be moved by brushing or by the

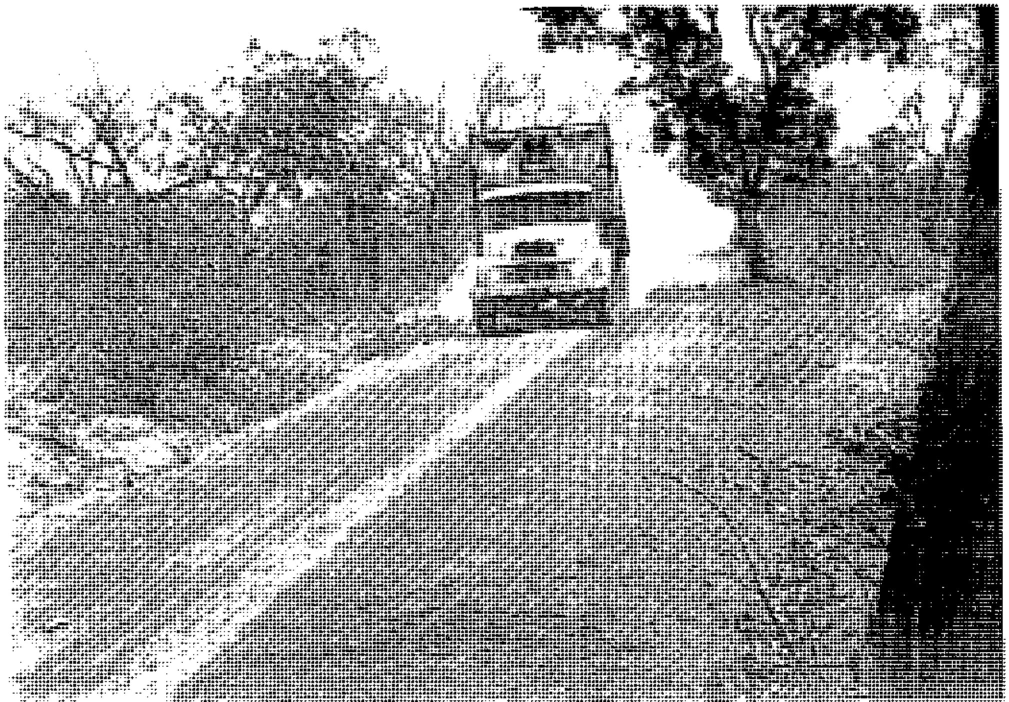


Fig.12.1 A typical corrugated gravel road.

light drags often used for gravel road maintenance. A motor grader or a heavy tractor-towed grader is required. And if, in the dry season, the material removed from the crests is left loose on the surface, the corrugations will appear once more within a few days. Heavy grading is work for the wet season and, to delay the onset of fresh corrugations, the material should be recompacted to a dense smooth surface.

A further observation may be of use in leading to an understanding of how corrugations form. With traffic travelling only in one direction, the corrugations move in a direction contrary to the traffic flow. Fig. 12.2 shows an example of this effect observed during a period of road reconstruction when traffic of the order of 150 vehicles per day was using the old road in one direction.

Clearly the formation of corrugations is related to the amount and speed of traffic. In a review of the literature, Heath and Robinson (1980) report wide acceptance of the proposition that corrugations are formed by forced oscillations at the resonant frequency of vehicles' suspension and tyre system. But there are variations in these frequencies over the range of vehicles using a road, and this does not explain why some materials corrugate and others do not.

There are many other examples of corrugations. Relton (1938), in a classic and entertaining paper, pointed out that wave formations are a common natural phenomenon. They occur on

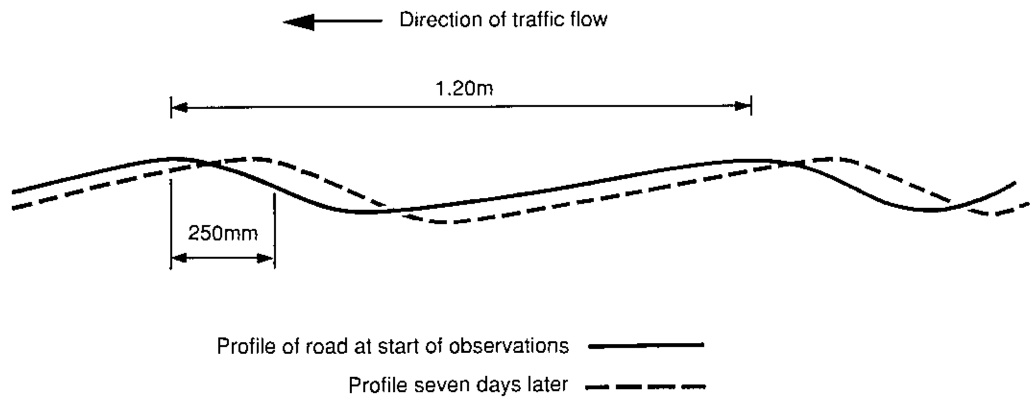


Fig.12.2 Movement of corrugations under one way traffic.

railway lines and can be seen at any busy railway terminus. Their pitch is of the order of 30 millimetres and appears to remain constant over the whole of the braking distance. They can occur in bituminous road surfacings, particularly where there is heavy braking. They are evident in the sand exposed by the retreating tide on the sea shore, and corrugations of similar pitch (150-200 millimetres) are seen as horizontal striations on sand dunes. Relton's example of the effect of a bow being drawn over a violin string provides a vital clue. The speed and pressure of the bow can produce variations in the intensity of the note. But the pitch of the note is entirely determined by the tension in the violin string. Corrugations on gravel roads are generated by traffic but their pitch and the rate at which they form are a function of the nature of the material. A graphic example from Ethiopia has been reported by Newill *et al* (1987). Coarse volcanic cinders occur in deposits surrounded by fine-textured soil deriving from volcanic dust. The cinders are an obvious choice for road gravelling in the regions where they occur. But in dry weather they corrugate very rapidly under traffic. As is shown in Fig. 12.3, the mixing of a small proportion of the fine volcanic soil with the cinders produced a ten-fold improvement in their ability to resist the formation of corrugations.

Choices of gravel for surfacing roads are always limited to the materials easily available in the vicinity. To control the rate at which corrugations form, the aim should be to select material most closely conforming to the gradings and plasticity characteristics defined for gravel surfacings in Table 12.1. Thereafter, loss of gravel and the formation of corrugations under traffic are minimised by maintenance during which the gravel surface is reshaped and well compacted.

12.5 Maintenance

Two extra tasks beyond normal routine road maintenance are needed on earth and gravel roads. One consists of periodic dragging and grading to preserve a tolerably even running surface. The other involves the provision of fresh material to replace material lost under the action of traffic

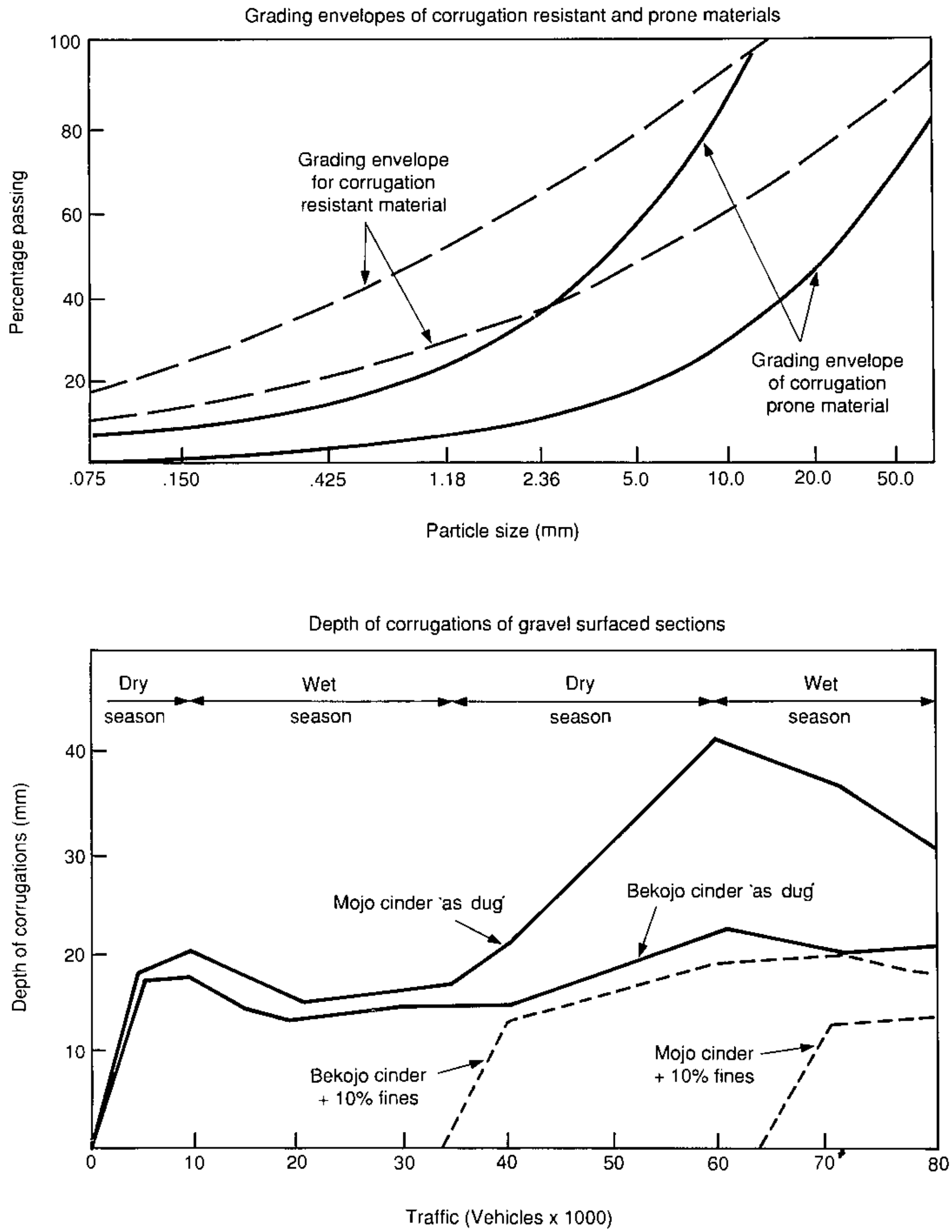


Fig.12.3 Effect of mixing fine soil with cinder gravel on the development of corrugations.

and weather. Dragging is a dry season task using brooms or tolards to distribute loose material evenly over the surface of the road and to remove excess loose material to the sides of the road, so reducing the rate at which corrugations can develop. Some typical tractor-towed drags are illustrated in Figs. 12.4 and 12.5. Bushes and small trees are sometimes used as drags but they are not very effective. Grading is a wet season operation using either blade-graders or tractor-towed graders to restore the running surface. The road surface is loosened to the depth of any corrugations, ruts and potholes. Any material that can be salvaged from the side of the road is brought in and the loosened mixture is spread evenly to the correct camber over the road surface. This loose material may then be left to be compacted by traffic, but, with materials that are prone to corrugate, the rate at which corrugations form will be greatly reduced by the use of a roller following on the operations of the grader. If grading is necessary in dry weather, for example, to remove heavy corrugations, it is always desirable to recompact the loosened material. Copious supplies of water will then be necessary. Regravelling is necessary at intervals depending on the rate of gravel loss. It becomes an urgent task when the thickness of gravel remaining over the soil is reduced to 75 millimetres.

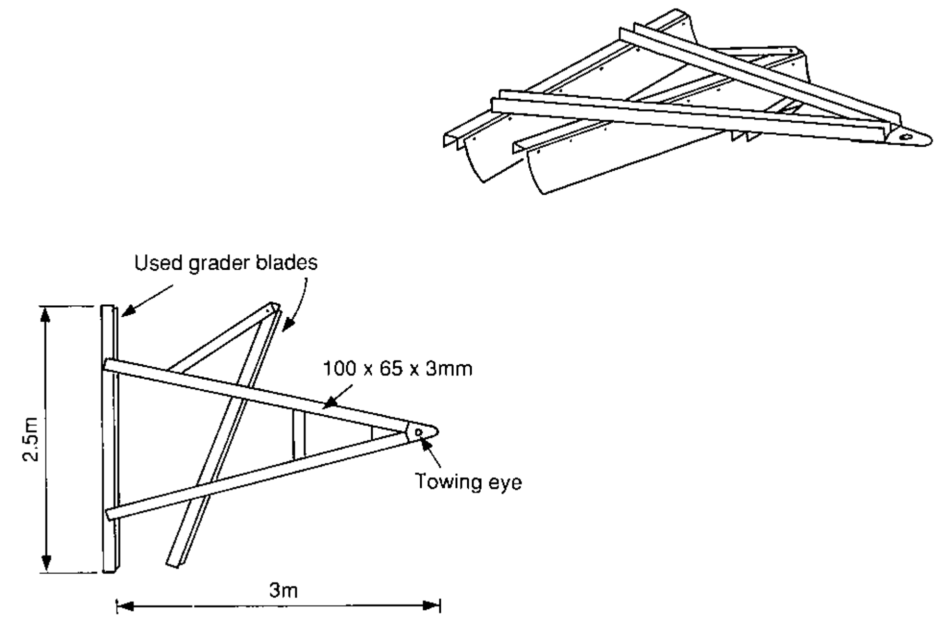
All highway departments are now seeking to put the planning of gravel road maintenance on a rational basis at district and national level. Frequencies of grading are clearly related to traffic intensity but they also reflect the nature of the road making gravels and local climatic conditions. Data on optimum grading frequencies derived in five countries, shown in Fig. 12.6, reflect these differences ((Jones (1984c) and Jones and Robinson (1986)). They probably also reflect some differences in the levels of road roughness associated with the optimum grading frequencies.

It is possible to use road investment models such as those developed by the World Bank (Watanatada *et al*, (1987) or the TRRL (Parsley and Robinson (1982)) to search for optimum levels of grading which minimise the sum of road maintenance and road user costs. Such methods have the advantage of producing economic justification for particular courses of action, but to employ them requires good knowledge of the rates at which road profiles deteriorate under traffic with the local gravels available and in the different climatic zones of a country. Such knowledge is emerging in many countries. In the meantime it may be necessary to use less sophisticated methods. Fig. 12.7, also from Jones and Robinson (1986), shows a simplified form of determining grading frequency and one objective in any country should be to build up knowledge of the typical ranges of road roughness associated with increasing levels of service.

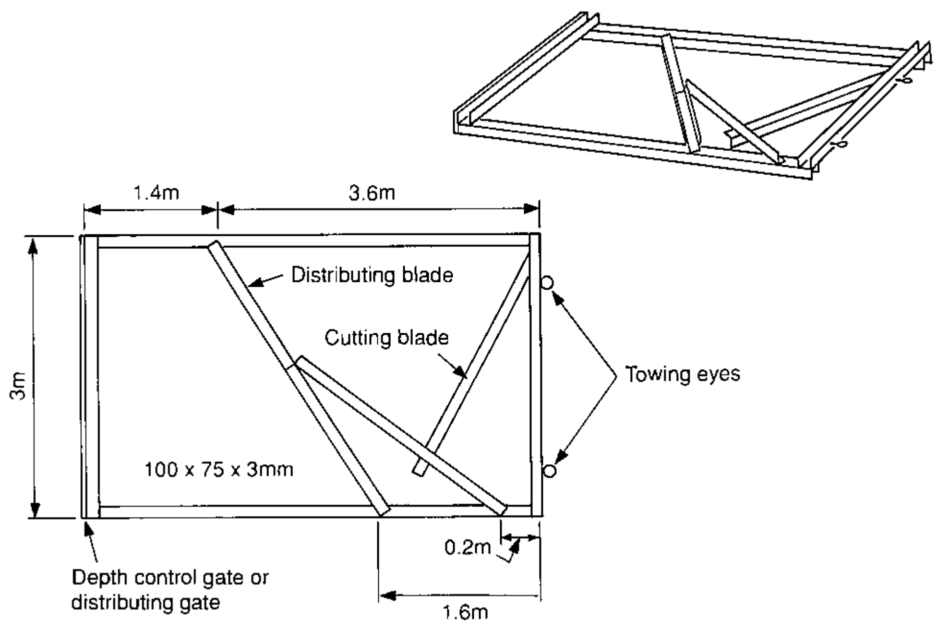
The rate of gravel loss is also affected by the nature of the gravel and the prevailing climate. Fig. 12.8 illustrates how losses of lateritic gravel can be affected by variations in annual rainfall. The planning of regravelling operations requires the building up of local knowledge on how traffic and climate affect the rate of loss from the road surface of the locally available gravels. Without such local knowledge the value of optimisation with road investment models is somewhat illusory.

12.6 Dust control

In dry weather, dust raised by traffic from unpaved roads is unpleasant to say the least. The reduction in visibility is a contributory factor in road accidents, the loss of material from the road

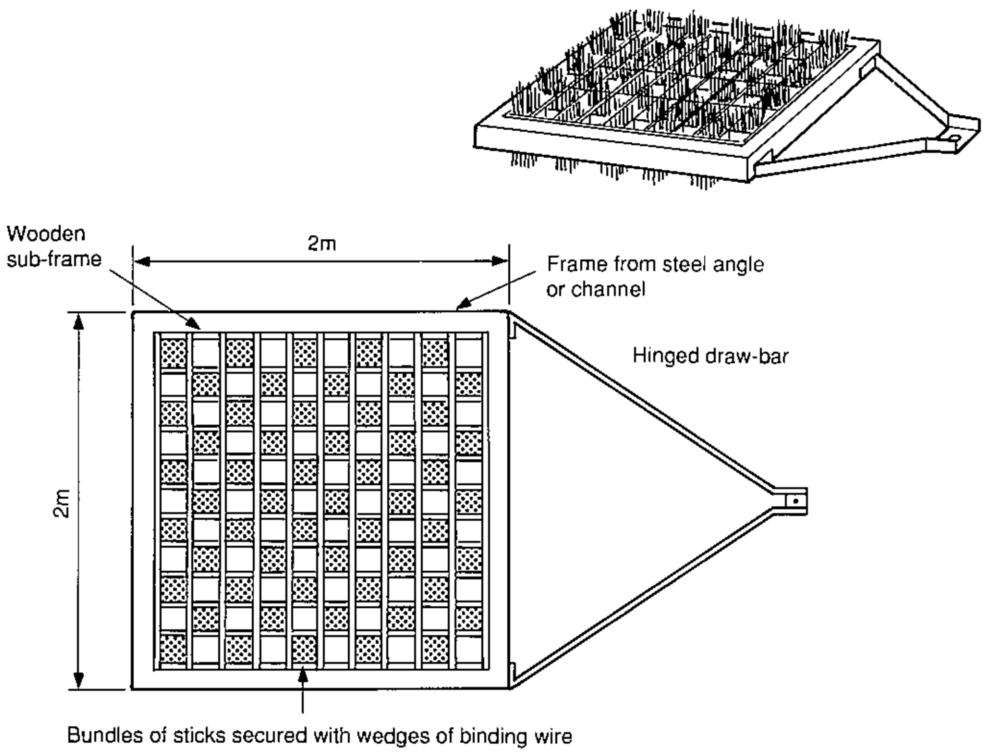


(a) 'A' FRAME DRAG UTILISING GRADER BLADES

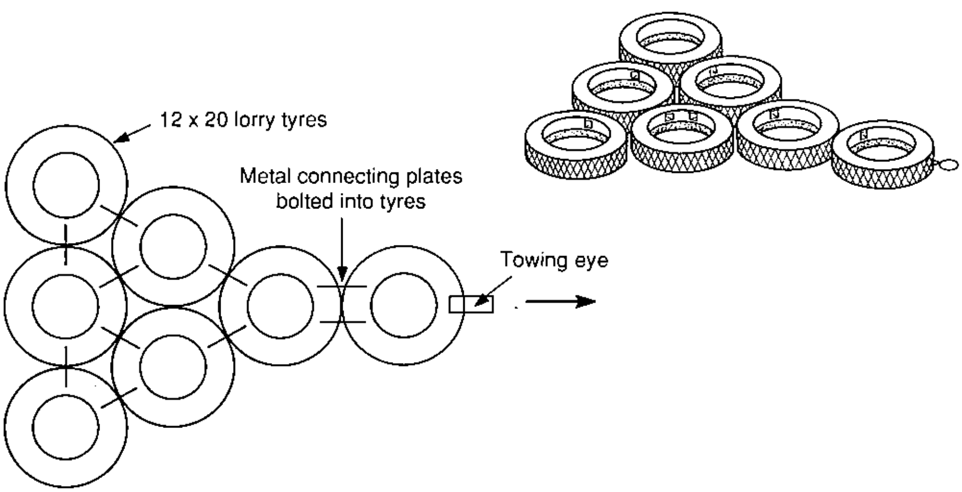


(b) RECTANGULAR TRIPLE BLADED DRAG

Fig.12.4 Examples of metal-frame cutting drags. (TRRL, Overseas Road Note 2)



(a) BRUSHWOOD DRAG



(b) TYRE SLEDGE

Fig.12.5 Examples of brushwood and tyre drags. (TRRL, Overseas Road Note 2)

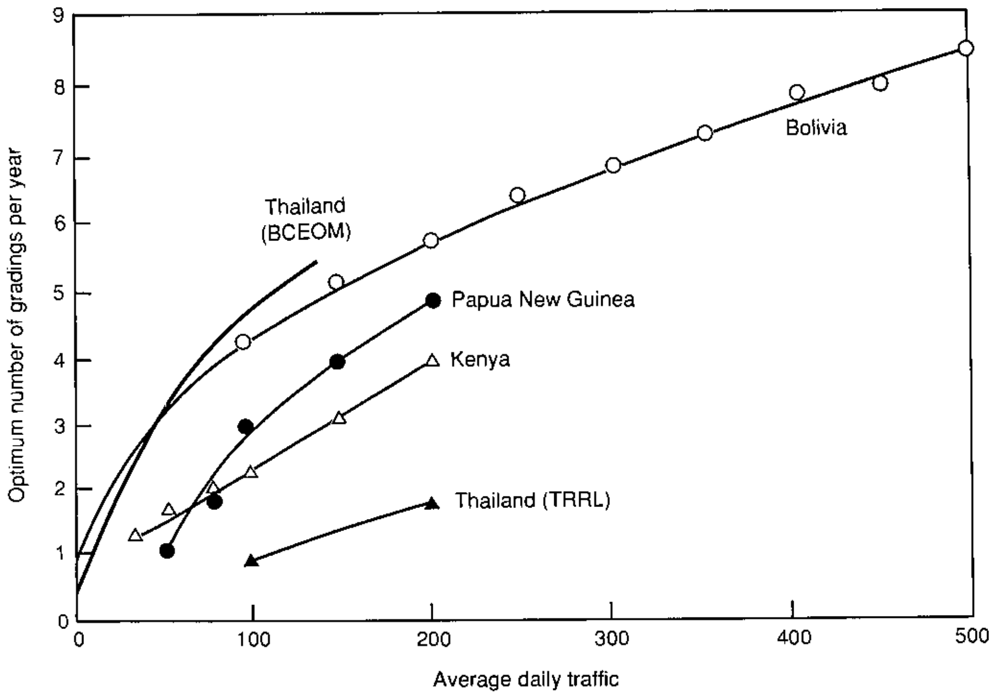


Fig.12.6 Optimum grading frequencies for gravel roads (depends on gravel, climate and traffic characteristics).

has economic consequences and, where unpaved roads pass through populated areas, the dust can be an intolerable nuisance. All these are relevant factors when calculating the benefits of providing dust free bituminous or concrete pavements.

In villages, some relief can be obtained by spraying the roads with water in the early morning but this relief is only temporary. However, there are compounds which can be applied to the road which are more durable in their effect. There are three types,

(a) deliquescent salts
(b) various organic compounds
(c) mineral oils.

Deliquescent salts function by retaining moisture in the surfacing. The best known is calcium chloride. It retains its deliquescent qualities provided that atmospheric humidity remains above 40 per cent. Its other limitation is that, being soluble in water, it is removed from the surface by rain.

Some of the organic compounds are available as industrial wastes. These function by coating the dust particles. They include sulphite liquor from one process of paper making, waste molasses from sugar cane, palm oil, and other vegetable oils. All are effective to some degree. Like other

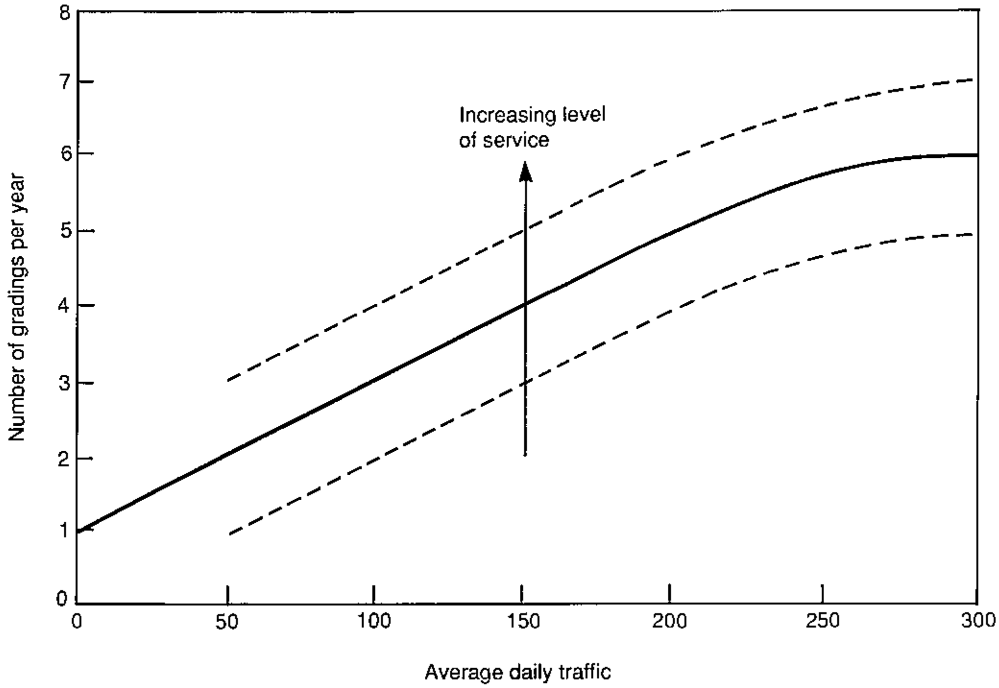


Fig.12.7 Recommended range of grading frequencies.

organic matter they decay quite rapidly in a tropical climate and their dust laying properties persist only for a year or so or until the first regrading operations are carried out.

Mineral oil may be available as waste sump oil or as bunker-fuel oil. Quite heavy applications are needed, sufficient to penetrate the surface for at least 20 millimetres. Their use delays the onset of potholes and corrugations but the surfaces tend to be slippery when wet, and their effect is lost as soon as regrading becomes necessary. Their use is likely to be confined to more lightly trafficked roads.

All dust treatments are only temporary in their effects and, when the costs of repeated applications are taken into account, they are likely to be more expensive than a more permanent treatment by surface dressing. They may, however, be a useful expedient whilst waiting for the more permanently surfaced road to be provided.

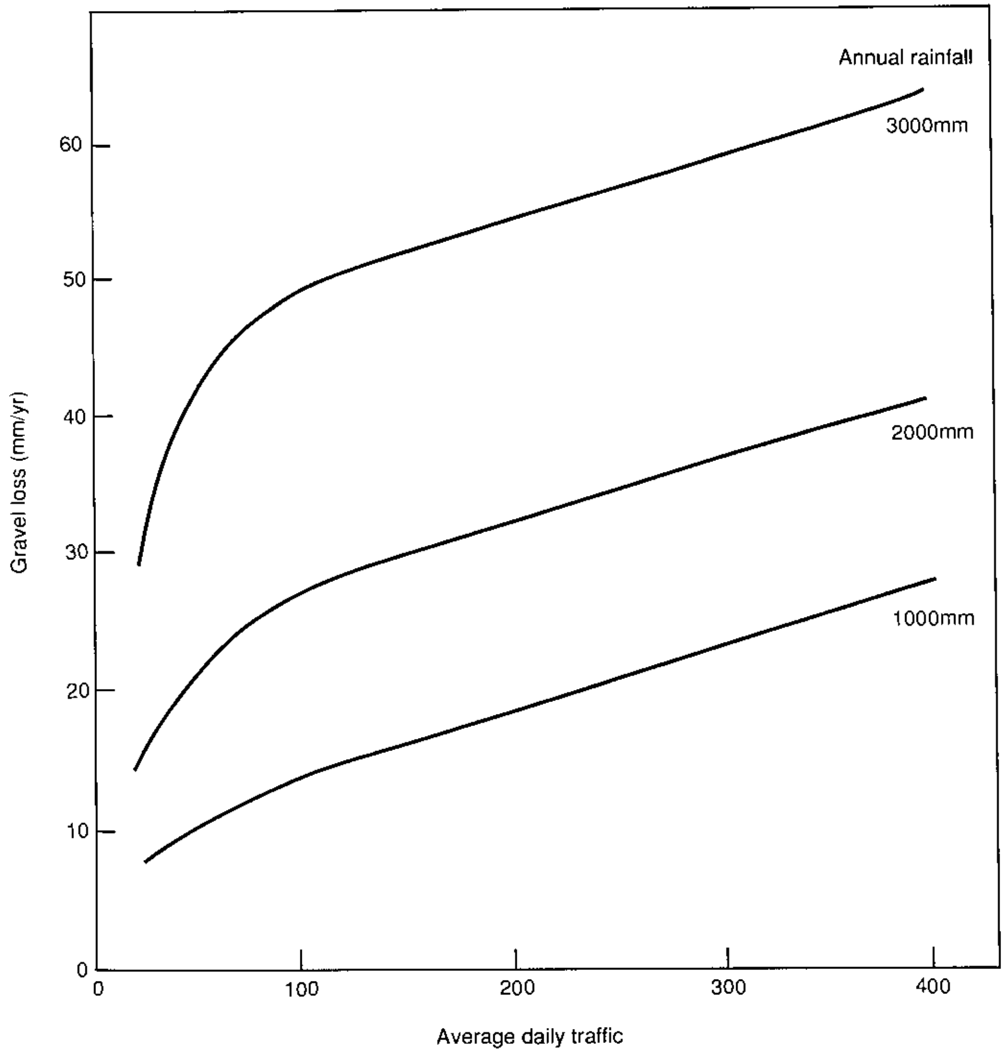


Fig.12.8 Gravel loss prediction curves for laterite gravels (Robinson, 1988).

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Devising methods of design and construction to provide an effective road system suitable for the needs of tropical countries has proved difficult, and considerable research has been necessary to understand the nature of the materials available, and the behaviour of the road structures themselves. This book has been written on behalf of the Overseas Centre of TRL to provide a comprehensive source of information for engineering students and highway engineers practising in hot, tropical countries. The first part deals with tropical soils and roadbuilding materials, describing their formation, characteristics and methods of testing. The second part describes the methods used to design and build roads, and includes chapters on all of the different types of pavements, including both concrete and unpaved roads.

The Overseas Centre (formerly the Overseas Unit) is financed by the Overseas Development Administration as part of the United Kingdom's aid programme for developing countries and much of the information contained in the book derives from the research carried out by the Overseas Centre on behalf of ODA.

Dr Millard's career as a highway engineer spans some fifty years. He first joined the Road Research Laboratory in 1943, and in 1954 became leader of the Tropical Section (now the Overseas Centre), a group formed by the British Government as a means of providing technical assistance on roads and road transport. Later, as Deputy Director of what had become the Transport and Road Research Laboratory, his main concern was with work on highway engineering. He then joined a firm of consultants as Resident Partner in the Far East, and became closely involved in projects backed by aid from the World Bank, eventually joining the organisation in 1976 as Highway Engineering Adviser. After his retirement he continued as a consultant to the World Bank, and to other organisations concerned with roads and road transport in hot countries.

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