LIMESTONE ROADS

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PREFACE

EXCELLENT textbooks have been written dealing with the many varied uses of Limestone, and a great deal has been written concerning road-surfacing materials and road construction generally. Little information, however, is available on the specific use of Limestone as an aggregate for road surfacings.

An attempt has been made in this book to give the fullest information possible in connection with the characteristics and properties of Limestone used as roadstone, and of Limestone road-surfacing materials.

In addition, information is given concerning the laying of the different types of road-surfacing materials described, in the hope that such notes may be of interest not only to the producer but to the consumer as well. It is only by an appreciation of the requirements of both, and by collaboration between the two, that progress is possible. For the same reason, brief notes on the manufacture of various forms of Limestone surfacing materials are included.

Unless specified to the contrary, the use of the word " limestone " in this book refers to a good-quality carboniferous mountain limestone.

My thanks are specially due to Mr. D. H. Gwinner, A.R.S.M., F.G.S., for writing Chapter I (" Notes on the Geology of Limestone ") and to Mr. J. Singleton-Green, M.Sc., A.M.I.Mech.E., M.I.Struct.E., M.Inst.Q., for preparing Chapter IX (" Concrete Roads "). The basis of Mr. Singleton-Green's chapter is his recently published book *Limestone Concrete* (Chapman & Hall) from which many passages have been copied verbatim.

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P. A. VALTON.

FROME, 1945.

PREFACE TO SECOND EDITION

B.S. 594 (Rolled Asphalt Hot Process) and B.S. 596 (Mastic Asphalt) have been revised and Chapter VIII has been re-written in order to give full weight to these new requirements.

I am very grateful indeed to several readers who have pointed out certain errors and omissions which have been rectified in this edition.

P. A. VALTON.

FROME, DECEMBER 1946.

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CHAPTER I

NOTES ON THE GEOLOGY OF LIMESTONES

by D. H. GWINNER, A.R.S.M., F.G.S.

BEFORE discussing the use of limestones as road aggregates, it is advisable briefly to consider how limestones were formed, as the characteristics of any material are determined by its mode of origin.

Classes of Rocks. The vast number of different types of rock, which form the crust of the Earth, and the source of so many and diverse raw materials for industry, can be divided into the following three main classes according to the general nature of the processes which attended their formation:

(1) The Igneous Rocks, such as granites and basalts, which had a fiery origin, and were formed by the uprising from deep-seated sources of molten rock material, which later cooled and solidified either on or below the Earth's surface.

(2) The Sedimentary Rocks, such as limestones and sandstones. This class comprises rocks formed by: (*a*) Animal and vegetable life, e.g., coal formed from vegetation and certain limestones formed from marine shells and other organisms. (*b*) The deposition on the floors of seas and lakes, or on land, by the agency of water, ice, or wind, of detrital material derived from the erosion of pre-existing rocks. (*c*) The precipitation by chemical or bacterial action of material carried in solution in water, e.g., certain limestones, deposits of rock-salt, gypsum, etc.

(3) The Metamorphic Rocks, such as gneiss, slate, and marbles, whose primary origin may have been either igneous or sedimentary, but whose original characters have been almost completely obscured by radical changes suffered through the effects of chemical action, heat, and/or pressure.

Origin of Normal Limestones. Limestones belong to the Sedimentary Rocks, and in their mineral content are much simpler than most rocks of igneous origin. Whereas in igneous rocks there are usually several different major mineral constituents, each playing its own part in influencing the character of the rock and its behaviour in use, normal limestones are composed mainly of one mineral—

calcite (the common crystalline form of calcium carbonate—$CaCO_3$). This mineral may be derived from any one of the following sources, or from a combination of any two or more of these sources:

(1) From the calcareous hard parts of marine and fresh-water organisms. On the death and decay of these organisms, their skeletons form the raw material of limestones. This is the commonest source of limestone deposits, and rocks composed mainly of such organic remains predominate among the calcareous deposits of most geological periods.

(2) From the precipitation of the chemical compound calcium carbonate from waters saturated with this salt. For the formation of large-scale limestone deposits by this means very special conditions are required involving high temperature, limited depth, and restricted circulation of the waters concerned.

(3) From fragments of pre-existing limestones, detached from the parent rock by weathering processes, transported by such agencies as river action, and deposited on the floors of seas and lakes.

In view of the variety of sources from which the principal raw material of limestones may be derived, it is not surprising that there are many types of limestone differing from each other in appearance and in properties.

Among the many varieties of organic limestones, some are named after certain organisms of whose remains they are largely composed. To mention but a few of these types, there are the so-called "Crinoidal Limestones" in which small plates of calcite, which once formed parts of the stems of crinoids (sea-lilies), are conspicuous; "Foraminiferal Limestones" crowded with the small shells of various foraminifera; and the "Shelly Limestones", consisting largely of the broken fragments of large shells together with the complete shells of smaller organisms cemented together in a mosaic of calcite crystals. There are also such varieties as "Coral Limestones", representing material which once formed coral reefs.

In any sea, the environment which governs the nature, variety, and abundance of the organisms whose life it supports, is determined by a combination of many factors such as the depth, temperature, salinity, and cleanliness of the water, the presence and strength of currents, and the nature of the sea-bed. Many of these factors, however, vary from one part of a sea to another. In some areas the sea will be colder and deeper than in others; at some points there will be currents scouring the sea-bed, and, particularly in coastal areas, we may expect the water to be not only relatively shallow but also to be more or less polluted by material from the land.

Rivers transporting land debris dump their loads when their velocity of flow is checked on their meeting the seas into which they discharge. The heaviest material carried is dropped first, lighter material is carried further from land, while the finest silt may be transported many miles out to sea before falling to the sea-floor, there to become mixed with such organic remains as are accumulating.

It follows, therefore, that near land from which rivers are draining, the conditions in a sea may be unfavourable to the presence of an abundance of the types of marine organisms whose skeletons form the raw material of organic limestones, and also that in such circumstances the admixture of land-derived material will in most cases be inimical to the formation of the purer types of limestone deposits. In such parts of a sea-bed, deposits of argilaceous (clayey) and arenaceous (sandy) rocks will be the general rule, and this is the principal reason for beds of limestone showing lateral variation when traced across country and of merging gradually into other types of rocks such as shales (i.e., consolidated clays) and grits and sandstones. This pollution of the sea by land-derived material is also one of the main causes of the presence of such impurities as silica (quartz), alumina (the principal mineral of clays), and iron in limestones.

Consolidation of Limestone Deposits. The processes of deposition of sediments are slow, and immense periods of time were involved in the accumulation of even the thinner deposits of limestone now quarried. Over such long periods, the conditions of deposition did not remain constant, and were often radically altered by changes in the relative levels of land and sea brought about by movements of the Earth's crust.

These movements, affecting the distribution of land and sea, have proceeded continuously throughout the ages, but at some periods to a greater degree and faster in some places than in others. By their action the floors of oceans have been raised to form the hills and mountains of land masses, while, conversely, land areas have sunk and been submerged to give rise to new seas and oceans.

When the level of a sea-floor is alternately raised and lowered by earth movements, the nature of the material accumulating on it will vary accordingly. Periods of such instability, accompanied by alternate deepening and shallowing of a sea, may give rise to deposits of limestones alternating and interbedded with shales and sandstones.

The vertical variations in the quality of limestones, noticeable in most large exposures, are often indicative of changes in the conditions of deposition caused by earth movements. Where great thicknesses of limestone occur showing little vertical variation, this indicates that the conditions in that part of the sea in which the deposit was formed remained fairly constant over a very long period.

Limestones were in an incoherent condition when they were first deposited on the floors of seas and lakes, and consisted of accumulations of shells and calcareous muds, but many have since been converted into hard and compact rocks. This lithification has been brought about by numerous complicated and interrelated processes, such as compaction and recrystallisation, which it is impossible to consider within the confines of this chapter. Deep burial beneath later sediments, together with the effect of the forces attendant upon earth movements, play an important part in converting the original incoherent deposit into a rock sufficiently hard and durable for use as road aggregates.

It follows that, in general, the oldest sediments, through having usually been the deepest buried, are likely to be represented to-day by the most compact and hardest limestones. This generalisation is not true in every case, but on the whole the limestones best suited for use as road aggregates are members of the older rather than of the younger geological formations.

Before proceeding to consider the types of limestone present in the various systems of rocks exposed in this country, it is necessary to digress in order to describe the nature of two special and common types of calcareous deposits—Magnesian deposits, in which the chemical compound magnesium carbonate ($MgCO_3$) plays an important part, and Oolitic limestones.

Magnesian Deposits. The chemical compound magnesium carbonate ($MgCO_3$) is an important primary constituent of the shells of many marine organisms, but analyses of modern organic calcareous sediments now in process of deposition show that the shells of tropical species of marine organisms contain more magnesium than those of their counterparts in cooler regions. It is natural therefore that magnesium carbonate should be present in many organic limestones, and in fact it may be present in any proportion even exceeding 50 per cent. of the rock.

The mineral forms in which magnesium carbonate exists in calcareous sedimentary rocks vary. In those in which the magnesium and calcium occur in approximately equal proportions, it is usually in chemical combination with calcium carbonate in the form

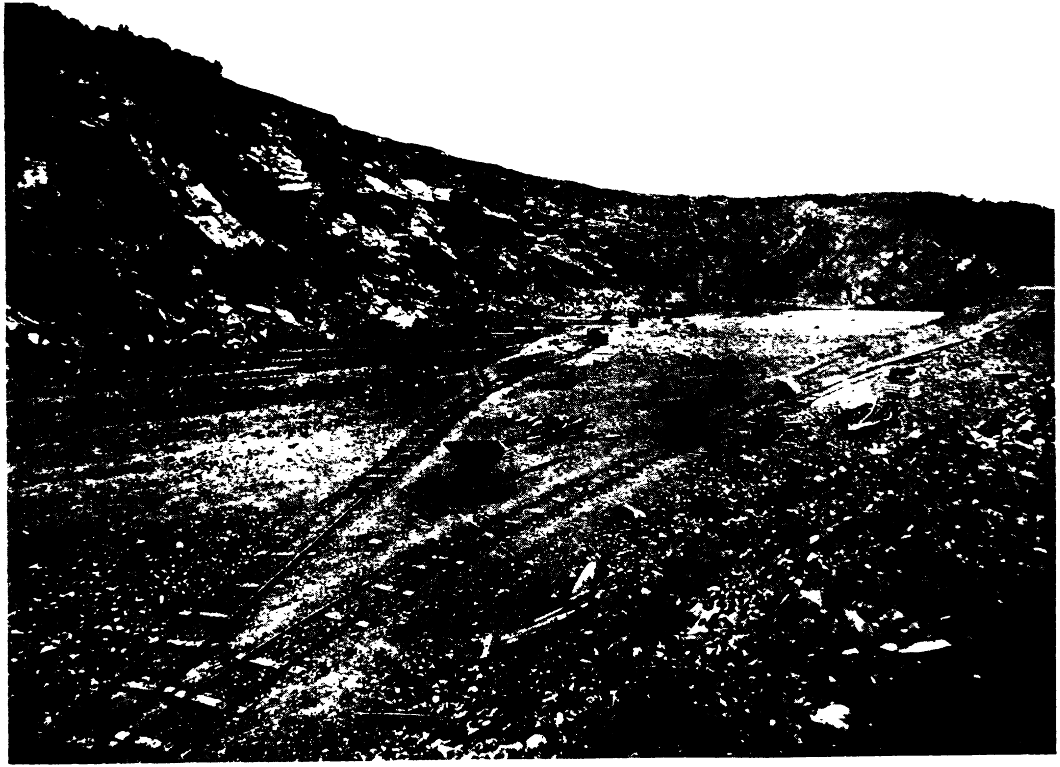

Fig. 1.—A Typical Limestone Roadstone Quarry.

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Fig. 2.—A Limestone Crushing and Screening Plant.

[Facing p. 5.

of the double carbonate of magnesium and calcium, i.e., as the mineral dolomite ($CaCO_3$–$MgCO_3$). In less magnesian types, and particularly in those containing less than 15 per cent. of magnesium carbonate, this compound may exist either in solid solution in the mineral calcite or as independent crystals of the mineral dolomite. Magnesium carbonate is, however, rarely present in sedimentary rocks in the form of independent crystals of the mineral magnesite ($MgCO_3$).

Confusion frequently arises in the terms used to describe various types of calcareous sedimentary rocks containing notable proportions of magnesium carbonate in its various forms. For example, the term " Magnesian Limestones " is best employed for those limestones which do not carry the mineral dolomite but do contain magnesium carbonate in solid solution in calcite. The term " Magnesian Limestones " is, however, often used as a label for the well-known Permian limestones of this country, most of which would be better described either as " Dolomitic Limestones " in those cases where the minerals calcite and dolomite are both present as independent crystals, or as " Dolomite Rock " in those cases where dolomite is the predominant mineral.

Magnesian and dolomitic deposits vary considerably and can originate in several different ways. In some types the magnesium carbonate was present in the sediment at the time of its original deposition, but in most it was acquired later, and in these is almost always present as the mineral dolomite.

Some of the types in which the magnesium carbonate was present initially were formed by the precipitation of magnesium compounds from the waters of saline lakes or inland seas, where the circulation of the waters was very restricted. It is believed that many of the dolomitic limestones and dolomite rocks which occur in the Permian System of this country originated in this manner.

In those more frequent types where the dolomitisation occurred later than the deposition of the sediment, this has arisen by the attack of magnesian solutions, and there are many examples where such attack has not produced a completely dolomitised rock. In these cases, the finer-grained material generally succumbs to dolomitisation more readily than the compact, coarsely crystalline material, and we find limestones in which the fine-grained ground-mass of the stone has been dolomitised while the coarsely crystalline fossil contents have resisted attack and remain as calcite.

Many of the bedded dolomitic limestones of considerable extent met notably in formations of Carboniferous age, appear to have been dolomitised by the effect of magnesian solutions shortly after the deposition of the calcareous sediments concerned; but where limestone of normal type is found suddenly to merge into dolomitic material, and where veins of dolomite rock occur in normal limestones, such effects can usually be ascribed to the percolation of magnesium-bearing waters from above finding their way into the limestones by fault planes and fissures at times long after the deposition of the limestones affected.

Dolomite being a slightly harder mineral than calcite, it might be expected that limestones of dolomitic type would all be harder than normal limestones and therefore more suitable for use as road aggregates. In some circumstances, however, some shrinkage occurs when a calcareous deposit is dolomitised, and in these cases a cavernous and porous structure is produced which weakens the resultant rock.

It is, therefore, impossible to judge merely on the evidence of the mineral dolomite in a limestone, or from the evidence of a chemical analysis showing a high proportion of magnesium carbonate, whether such a rock will prove more or less suitable for use as a road aggregate than a more normal limestone, for so much depends on other factors. For example, the Permian limestones of this country, many of which are highly dolomitic, are on the whole much inferior to Carboniferous Mountain Limestones for such a purpose, being more porous and weaker; on the other hand, many dolomitised Carboniferous Mountain Limestones are as good for use as road aggregates as Carboniferous Mountain Limestones of normal type.

Oolitic Limestones. Oolitic types of limestone are largely composed of small spherical bodies called ooliths, usually about 1 millimetre in diameter, cemented together by a ground-mass which normally consists principally of calcite. These ooliths, which resemble the roe of a fish, derive their name from the Greek words *oon* (an egg) and *lithos* (a stone). These small, roughly spherical bodies are found on examination to consist of concentric layers of calcite (and in recent oolitic deposits of aragonite—another crystalline form of calcium carbonate) around nuclei consisting either of minute fragments of shells or small sand-grains.

It is believed that the general conditions which gave rise to such deposits consisted of very shallow water containing a high concentration of calcium carbonate in solution. Vigorous movement of the water, by tidal currents or wave action, kept the nuclei in a state of continual movement during their encasement with calcite, and accounts for the roughly spherical shape of the ooliths.

Oolitic deposits are found among the calcareous sedimentary rocks of most geological ages, but attain their greatest prominence in the Jurassic System, where they are represented by some important types of building stones, such as the well-known Portland Stone.

Distribution of Limestone Deposits. The rocks exposed and comprising the outer parts of the Earth's crust have, for purposes of reference and description, been divided on the basis of their fossil contents and interrelationships into a number of systems. These are listed below in descending order of increasing age:

TABLE I

Table showing Names given to Periods of Geological Time, and the Systems of Rocks formed in each

Era	Period and system
Cainozoic	Recent
	Tertiary
Mesozoic	Cretaceous
	Jurassic
Palaeozoic	Triassic
	Permian
	Carboniferous
	Devonian
	Silurian
	Ordovician
	Cambrian
Archaean	Pre-Cambrian

The folding sketch-map of England and Wales (Plate I), based on the map entitled " Map of the Chief Limestones of England and Wales ", issued by the Geological Survey of Great Britain, roughly indicates the areas occupied by outcrops of the more important calcareous sedimentary rocks of certain geological systems. The scale of this map is so small that it must be accepted as only a very general guide to the distribution of these rocks. Where, for example, an area on the map is so shaded as to indicate the presence of mountain limestones, it must not be assumed that all the strata outcropping in the area so marked are necessarily of this type. For more accurate and detailed information regarding any particular area, reference should be made to the geological maps on scales of 1 inch to the mile and 6 inches to the mile prepared by H.M. Geological Survey, and also to the relevant Geological Survey Memoirs.

In briefly describing the principal limestones of England and Wales, it is convenient to consider each geological system separately:

Principal Limestone Deposits of England and Wales.

Pre-Cambrian. This system, comprising the most ancient rocks, has been subjected to so many and to such intense earth movements that most of its limestones have either been altered into marbles or are so shattered that they are too weak for use as road aggregates.

Cambrian. Limestones are scarce in this system. The seas which covered parts of England and Wales in the Cambrian Period were much polluted with land debris swept into them from neighbouring land-masses, and conditions were therefore unfavourable to the formation of limestones.

A further cause of the scarcity of limestones in these rocks is that in the seas of that period few of the marine organisms had shells and hard parts of calcite, their skeletons being composed of a horny chitinous material.

The system is well exposed in Wales, but most of its sediments are now represented by slates, sandstones and grits. There are, however, outcrops of thin and unimportant Cambrian limestones in Shropshire and also in the Midlands.

Ordovician. Although deposits of Ordovician limestones are more extensive in England and Wales than those of the preceding Cambrian System, they are mostly impure types and of little economic importance.

During the Ordovician Period there were many earth movements and considerable volcanic activity. In consequence, the limestones of this period vary considerably from one district to another, and generalisations on their nature and quality are difficult. Many of the limestones are sandy and were laid down in shallow water as masses of lenticular shape thinning out in all directions. Both lateral and vertical variation is pronounced, and some of these limestones weather very badly (e.g., the Coniston Limestone of the Lake District).

Ordovician limestones suitable for use as road aggregates are rare and of only local occurrence.

Silurian. While the Silurian Period in England and Wales was marine, like the Ordovician, organisms possessing shells and hard parts of calcium carbonate were much more abundant in this than in the earlier geological periods. In consequence, limestones were formed extensively in the Silurian Period.

The system includes such limestones as the Wenlock, Woolhope, and Aymestry Limestones. They vary fairly considerably in quality from one locality to another, sometimes being hard and crystalline and not unlike the Carboniferous Mountain Limestones, and sometimes earthy and nodular.

Silurian limestones suitable for use as road aggregates are rather restricted in occurrence.

Devonian. In the Devonian Period a sea covered the south-western counties of England, while a lake or estuary covered South Wales. Except in the southern sea, few important deposits of calcareous sediments were formed.

In the estuarine or lacustrine area there accumulated thick deposits of marls, sandstones, and conglomerates, mostly of a red colour, which have been named the Old Red Sandstone. The nature of these deposits indicates that the conditions at the time of their formation were arid and desert-like. Limestones are rare in this series, but the so-called " Cornstone Limestones " occur as part of the Old Red Sandstone in South Wales and the border counties as thin lenticular masses of little economic importance.

The Devonian marine rocks, formed in the southern sea, include extensive and thick deposits of limestone in which the remains of corals and crinoids (sea-lilies) are conspicuous. These limestones show considerable variation in mechanical strength when traced from one locality to another, for in some areas they have been crushed and weakened by the pressure which accompanied the formation of the slates with which they are frequently associated. Some of these Devonian limestones are extensively dolomitised.

Although there is much variation in the properties of the Devonian limestones, they include extensive masses of stone suitable for use as road aggregates, and these are similar in many respects to some of the Carboniferous Mountain Limestones of the succeeding system.

Carboniferous. The rocks of the Carboniferous Period include the most important and the most extensive deposits of hard limestones in this country.

During the early part of this period, the mountain-building earth movements and arid desert conditions of the Devonian gave way to a long phase of slow and gentle subsidence, and apart from areas of high ground which remained as islands, the whole of England, Wales, Ireland, and the south of Scotland became submerged by an extensive sea.

Except in the extreme north and south, very little land-derived material was carried into this Carboniferous sea during the long period of subsidence, and the environment was therefore favourable to the formation of limestone deposits.

The subsidence, mentioned above, began first in the south, and the earliest and oldest Carboniferous limestones which were deposited in the south do not therefore occur in the northern areas which were not submerged until later. Enormous thicknesses of limestone were laid down in this Carboniferous sea, over 3,000 ft. being formed in the Mendips and the Gower Peninsula, while in Derbyshire over 1,500 ft. of limestone beds are now exposed.

Such vast deposits of calcareous material, formed from a continual shower on to the sea-floor of the shells and skeletons of marine organisms, maintained the depth of the sea fairly constant over long periods. In general, the rate of accumulation of sediments on the sea-floor kept pace with the rate of subsidence. This state of affairs was, however, interrupted from time to time by upward movements which caused local shallowing of the sea, and even occasional temporary emergence of the recently accumulated deposits above sea-level.

This occasional shallowing is one of the causes of the presence of numerous varieties of limestone in the Lower Carboniferous Series. The limestones include, apart from magnesian, dolomitic, and oolitic types, normal limestones of almost every degree of texture from the porcellaneous very fine-grained calcite mudstones, apparently formed under lagoonal conditions, to coarsely crystalline shelly varieties. In colour the limestones range from black through every shade of grey to almost white, but most are grey. Some of the black varieties are distinctly bituminous.

Dolomitic limestones are common in the Carboniferous, and as previously mentioned, most owe their dolomitisation to the action of magnesian solutions on the sediments after their original deposition.

In spite of the fact that the Carboniferous limestones include so many varieties, and that in some localities lateral and vertical variation is pronounced, the series as a whole contains vast quantities of high-quality, compact, and strong Mountain Limestones well suited for use as road aggregates.

Before the end of the Carboniferous Period the phase of general subsidence ceased, and gave way to an upward movement which terminated the long spell of limestone accumulation and caused the deposition of muds, sands, and grits in the Carboniferous sea. Later still, when the sea basin had become choked with sediments, low-lying mud-flats were formed, and on these flourished the forests which, after burial under younger sediments, gave rise to the Coal Measures to which the Carboniferous Period owes its name.

Permian. At the end of Carboniferous times the climate changed and became unfavourable to the support of the forests to which we owe our principal coal seams. This change was due to the initiation of a fresh phase of earth movements which, in the Permian Period, uplifted what had been low-lying swamps into dry land, and formed a chain of mountains to the south and south-west of Britain. These mountains acted as a barrier against the moisture-carrying south-west winds and so caused the climate of the area now occupied by England and Wales to be arid.

About the middle of the Permian Period there occurred a phase of subsidence whereby the northern parts of England were converted into a land-locked gulf, which later became a salt-lake. This lake gradually dried, causing an increasing concentration of the salts carried in solution in its waters, and finally in their precipitation and deposition as beds of salt and gypsum.

It will be appreciated that these conditions were most unfavourable to the support of life. The limestones, therefore, which were formed in this Permian island sea owe their existence far more to the chemical precipitation of calcareous and magnesium compounds present in the water than to the accumulation of organic remains.

These Permian limestones, often referred to as the " Magnesian Limestones ", range from highly calcareous types to dolomite rocks. They are very variable even locally, and in the space of a single quarry several very different varieties are often present. On the whole, they are much less compact than the so-called Mountain Limestones of the preceding Carboniferous Period, and are frequently sandy and porous. Not only was their origin very different from the majority of Carboniferous limestones, but the formation, being younger, has been less compacted.

Except for occasional local occurrences, these Permian limestones are seldom suitable for use as road aggregates.

Triassic. The conditions in England and Wales during the Triassic Period were generally similar to those obtaining in the Permian, but no important accumulations of limestone were formed.

Jurassic. The Jurassic Period in England and Wales was mainly marine, and its principal deposits consist of clays and limestones. Although many Jurassic limestones are useful as building stones, e.g., the well-known Portland Stone, they are too soft and porous for use as road aggregates. Most of these Jurassic limestones were laid down in shallow-water conditions and many are oolitic.

Cretaceous. During the Cretaceous Period a sea covered nearly the whole of this country, and, as in Carboniferous times, this great submergence was accompanied by the formation of thick and extensive calcareous deposits. These are now represented by the Chalk. This soft, earthy, pure limestone is the most extensive of all limestone deposits in this country, but not having been compacted and recrystallised to the same degree as the much older deposits of the Silurian, Devonian, and Carboniferous Periods, it is useless for roadstone purposes.

Tertiary. In the Tertiary Period there were very few limestones formed in this country. There are some thin and shelly Tertiary limestones (the Bembridge Limestone) exposed in the Isle of Wight, but these are of no interest as a source of roadstone, being limited in extent and too soft for such a purpose.

So far as their utility for roadstone purposes is concerned, the limestones of the various geological systems described above can be summarised as follows:

TABLE 2

Geological period	Usefulness of limestone for roadstone purposes
Tertiary	No limestones of importance in U.K.
Cretaceous	Chalk. Unsuitable, too soft.
Jurassic	Oolitic shallow-water types. Generally unsuitable, too soft and porous.
Triassic	No limestones of importance in U.K.
Permian	Mainly dolomitic types. Generally too weak and porous, but some local occurrences of suitable stone.
Carboniferous	Mainly compact crystalline Mountain Limestones. Very extensive deposits. Many varieties suitable for roadstone.
Devonian	Some compact crystalline Mountain Limestones. Much variation in mechanical strength owing to shattering by pressure in certain localities, but some occurrences suitable for roadstone.
Silurian	Some compact crystalline Mountain Limestones. Considerable variation in different localities. Some occurrences of suitable stone.
Ordovician	Limestones very variable in quality. A few restricted occurrences of compact crystalline Mountain Limestones suitable for roadstone.
Cambrian	No limestones of importance in U.K.
Pre-Cambrian	Marbles and limestones shattered by pressure. Unsuitable—too weak.

Relation between the Nature of a Limestone and its Suitability as a Roadstone. The standard tests in use for assessing the properties of roadstones are described in detail in a later chapter, which also mentions the results of such tests on limestones.

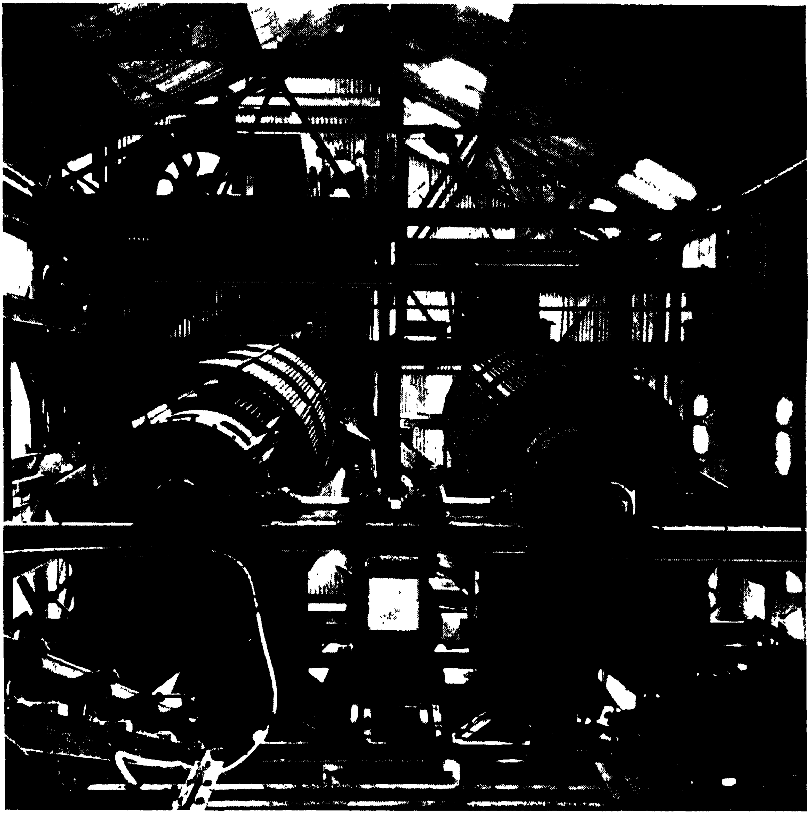

Fig. 3.—Rotary Screens for the Preparation of Sized Roadstone.

[*Facing p.* 12.

In considering the relative merits of different types of limestone for use as road aggregates, there is very little published data on which to base conclusions, for, in spite of the interest and importance of the subject, much less research work has been carried out on the relationship between the nature and behaviour in use of limestones than has been devoted to this aspect of the igneous rocks.

Since a limestone to be suitable for use as a road aggregate must be compact, have a high crushing strength, and give good adhesion with binders, the geological age of any limestone serves as a useful guide to its probable suitability. The information already given shows that most limestones formed since the close of the Permian Period are too soft and porous for use as roadstone, the older limestones being generally both denser and stronger.

Tests carried out by the laboratories of The British Limestone (Roadstone) Federation on samples of Mountain Limestones from 30 different quarries working the older formations, gave an average crushing strength of 27,800 lb. per square inch for such limestone, and 12 of these limestones gave results exceeding 30,000 lb. per square inch.

In general, the medium and finer-grained crystalline varieties of limestone are the strongest. Varieties containing a high proportion of large crystals of calcite, either in the ground mass or as parts of large fossils, are liable to be weakened by the presence of such crystals.

Other causes of weakness in limestones are the presence of veins and cracks, even though these may be filled with crystalline calcite. Such planes of weakness are common in those Devonian limestones which have been much affected by pressure, and are also met in certain calcite mudstones of Carboniferous age which apparently shrank considerably on drying. In some cases dolomitisation can give rise to such shrinkage cracks. While such planes of weakness are sometimes obvious in the hand specimen, their presence may not be apparent until the rock is examined in thin section under the microscope.

Colour by itself is an unsafe guide to the suitability of a limestone for use as a road aggregate, but there is a slight tendency for the darker to be on the whole rather stronger than the paler varieties.

Dolomitisation, as previously mentioned, is in itself no criterion of the usefulness of a limestone, for it may have either a toughening or weakening effect on the rock.

In general, therefore, the best varieties of limestone for use as road aggregates are the dense, medium, and fine-grained crystalline types, free from veins, cracks, and cavities, and not very pale in colour. While such varieties occur in the Ordovician and Silurian Systems, and more frequently among the Devonian Mountain Limestones, they attain their greatest prominence in the Carboniferous Mountain Limestones.

CHAPTER II

PHYSICAL AND CHEMICAL PROPERTIES OF LIMESTONE

Abrasion Test—Adhesion Test—Aggregate Crushing Test—Attrition Test—Crushing Strength—Impact Test—Water Absorption—Specific Gravity

ALL the recognised physical tests on roadstone described in this chapter, with the exception of the aggregate crushing test, were introduced in the days of water-bound macadam roads.

Modern fast-moving traffic has brought with it different forms of construction (surface dressing, tar or bituminous macadam, asphalt, concrete, etc.). All these materials must be considered as a mixture of stone and a binder, and it is not the performance of the roadstone under test in the laboratory, but the performance of the roadstone *as part of a mixture* which matters. No satisfactory tests on the actual mixture have been devised yet. The installation of circular testing tracks by the Road Research Laboratory was an attempt to meet this problem, but it is understood that difficulties were experienced in correlating the results obtained on large-scale tests under normal traffic conditions with those obtained on a small scale on the testing track under accelerated traffic and weathering conditions.

Nevertheless, the " N.P.L." physical tests are still in use, and a brief description of these tests with some notes on their significance and limitations is given below.

Abrasion Test. This test is carried out by pressing the base of a weighed cylinder of the stone, at a specified load, against a revolving metal plate upon which standard sand is fed. The cylinder is ground or cut from the specimen to be tested, to a size of 1 in. high by 1 in. diameter. The loss in weight of the specimen after 1,000 revolutions is estimated, such loss in weight representing the wear.

The test is fully described in B.S. 812,[1] where the so-called coefficient of hardness is defined as:

$$\text{Coefficient of hardness} = 20 - \frac{\text{Loss of weight in grams}}{3}$$

It will be seen from this formula that the maximum coefficient obtainable, assuming that no wear took place at all, would be 20.

[1] B.S. 812. Sampling and Testing of Mineral Aggregates, Sands and Fillers. (British Standards Institution.)

A hard limestone of the carboniferous mountain type gives satisfactory readings, but this test, devised in the days of horse-drawn traffic, fails to give a true indication of the wearing properties of the roadstone under modern traffic conditions, for the following two reasons:

1. Present-day traffic is carried almost exclusively on rubber tyres, and the abrasive effect of these is of a very low order compared to the iron-tyred traffic of many years ago.
2. In the case of tar or bituminous surfacings, the stone is completely covered by binder and this binder film has to be worn away before the traffic can have any abrasive effect on the stone aggregate. It is surprising how long such a film of binder persists on the top surface of a road before the stone is exposed. The writer has observed cases of heavily trafficked bituminous carpetings which, after many years, still possessed an appreciable thickness of bitumen on the top surface. This further amplifies the previous argument that the abrasive effect of modern traffic is relatively small. A modern road does not need replacement because the aggregate has worn away, but usually because of weakness of the internal structure. In the case of a tar or bituminous surface, this weakness is usually caused by a low adhesion value of the stone used.

Adhesion Test. Many tests have been devised to estimate the adhesive properties of binder and aggregate. Of these, the Riedel and Weber test, which is incorporated in the German Standard Specification DIN. 1995, gives reliable results and takes into account many factors which are omitted from other tests.

The stone used for the test is ground down, and only material passing B.S. 25 and retained on B.S. 72 sieves is used. Aggregate and binder are warmed to specified temperatures and 71 per cent. of the aggregate is mixed with 29 per cent. of the binder by volume. About ½ gram of this mixture is boiled in water for one minute in a test-tube. If the binder separates from the stone, the adhesion is poor and is classified as "0". If no separation occurs, a further ½ gram of the mixture is boiled with 6 c.c. of dilute sodium carbonate solution. Further ½-gram samples are treated with sodium carbonate of increasing concentration until separation of the binder from the aggregate occurs. The solutions are numbered from 0 to 9, Solution No. 9 consisting of normal (N/1) sodium carbonate (106 grams per litre). The strength of the other solutions are shown in Table 3. The number of the solution which causes separation represents the adhesion value of the particular stone and binder used. If no separation is effected by the strongest solution, No. 9, then the maximum adhesion value of 10 is recorded. Intermediate values are obtained according to Table 3.

TABLE 3

Riedel and Weber Constants

Solution number	Concentration of sodium carbonate solution causing separation	Power of adhesion	
0	0 (water)	0	Poor
1	N/256	1	Fair
2	N/128	2	
3	N/64	3	
4	N/32	4	Good
5	N/16	5	
6	N/8	6	
7	N/4	7	
8	N/2	8	
9	N/1	9	
	No separation	10	

In the case of some aggregates it is not possible to obtain a clear line of demarcation. For example, partial stripping may start with Solution No. 1 and it may be necessary to go up to Solution No. 3 to achieve complete stripping. Such a specimen would be recorded as having an adhesion value of 1–3.

Such a test is of the greatest importance in determining the suitability of an aggregate for tar or bituminous work. Hardness alone, without good adhesive properties, is useless, and the strength of a bituminous surfacing lies in its weakest link, which is undoubtedly the binder film, the main purpose of which is to hold the aggregate together.

Limestones have very high adhesion values towards tar or asphaltic bitumen. The Research Laboratory of the British Limestone (Roadstone) Federation has carried our Riedel and Weber adhesion tests on samples of stones submitted by Members of the Federation, and the average of 30 different limestones was 6·8 for tar and 8·8 for asphaltic bitumen. Slags generally have satisfactory adhesion values. Many igneous rocks and most gravels give poor readings.

The superiority of limestone over other aggregates has been amply proved in practice. Laboratory research has also convincingly established this fact. Dr. B. H. Knight [1] has published the following interesting table, which is reproduced by courtesy of the Institution of Municipal and County Engineers.

TABLE 4

Order of Merit of Adhesion Value of Aggregates (on Basis of Riedel and Weber Test)

Note.—No fillers were added to the binders in the tests

Power of adhesion	Order of merit	Type of aggregate	Adhesion value		
			Average	Maximum	Minimum
Good	1	Dolomite	10·0	10	10
	2	Bituminous limestone .	8·7	10	7
	3	Devonian limestone .	8·5	10	9
	4	Crystalline iron slag .	7·9	10	8
	5	Basic steel slag. . .	7·5	10	6
	6	Carboniferous limestone	7·2	10	3
	7	Iron slag, partly glassy	7·1	10	7
	8	Epidiorite	6·1	10	4
	9	Quartz sand . . .	6·0	9	4
	10	Oolitic limestone . .	6·0	10	0
	11	Chalk	5·4	10	0
	12	Baked dolerite . . .	4·5	10	0
Fair	13	Fine-grained granite .	1·4	8	0
Poor	14	Flint.	0·4	2	0
	15	Quartzite	0·2	1	0
	16	Porphyry	0·1	1	0
	17	Coarse-grained granite.	0·1	1	0

It should be noted that the description "bituminous limestone" in the above table does not imply that the limestone was coated with bitumen, but is a geological description of dark-coloured limestone containing traces of organic matter.

The importance of the adhesion test will be discussed further in Chapter III.

Aggregate Crushing Test. This test is also described in B.S. 812 [2] and has the advantage that actual samples of chippings

[1] Bernard H. Knight, "The Adhesion of Bituminous Binders to Road Aggregates". *Journal of the Institution of Municipal and County Engineers,* 1 March, 1938.

[2] B.S. 812. Sampling and Testing of Mineral Aggregates, Sands and Fillers. (British Standards Institution.)

as supplied for use can be tested. Essentially the test consists in placing a known weight of the chippings in a cylinder of standard dimensions (see Fig. 4). The cylinder rests on a smooth steel plate and pressure is applied to the sample at the top by means of a loose-fitting plunger. A load of 40 tons is then applied at a uniform rate of 4 tons per minute.

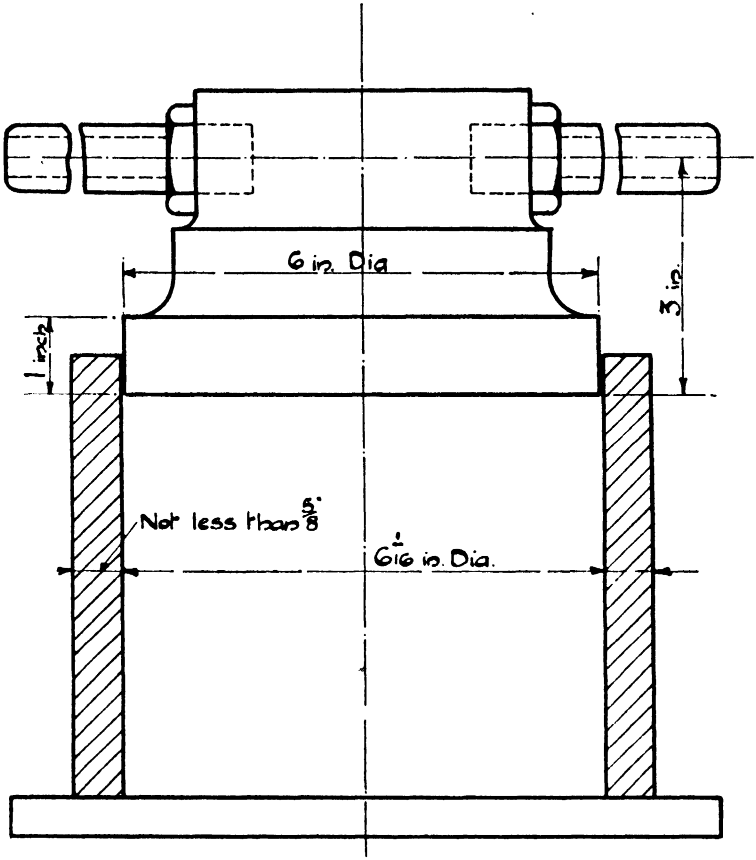

Fig. 4.—Mould for Aggregate Crushing Test.

After this pressure has been applied, the material is sieved on a No. 7 B.S. test sieve (mesh aperture approximately $\frac{1}{10}$ in.). The portion passing the test sieve is weighed and the aggregate crushing value is reported as a percentage, calculated as follows :

$$\text{Aggregate crushing value} = \frac{100 \times \text{Weight passing 7 mesh after test}}{\text{Weight of sample tested}}$$

This test is very severe, and aggregate crushing values lying between 10 and 25 per cent. have been obtained with accepted road-stones. Good Carboniferous mountain limestone will give values within this range. This test is being extended to cover also the macadam sizes of stone.

Attrition Test. A dust-free 11-lb. sample of stone, all passing 2-in. and retained on 1½-in. square mesh, is placed in a hollow iron cylinder closed at both ends, which is mounted on a shaft with its axis inclined at 30 degrees to the axis of rotation of the shaft. After 10,000 revolutions of the cylinder, the stone is sieved on a 10 mesh sieve. The loss in weight, expressed as a percentage of the original weight of stone taken, gives an index of the attrition value of the stone. A wet test is also carried out by adding an equal weight of water to the stone in the cylinder. Most stones show a greater loss by attrition in the wet test.

The same arguments against the value of this test, as outlined under "Abrasion Test", apply here even more strongly. Stone aggregate of the old water-bound roads was in a continual state of movement owing to the lack of strength of the dust mortar, so that a great deal of internal attrition, caused by stones rubbing together, was inevitable. It was this movement and attrition which caused the relatively rapid disintegration of these roads, with consequent creation of excessive dust. The stone was not only ground to dust by the direct action of the traffic, but also by the displacing effect this traffic had on the stones, with resulting internal movement.

A modern tar, bituminous, or concrete road is a very different proposition. Each individual stone is firmly bonded to its neighbours by means of a permanent and durable mortar (tar, asphaltic bitumen, or cement), so that internal movement of the structure can be almost disregarded. In fact, one can go further and state that a modern road would automatically fail if the binder employed allowed any such movement; individual stones would in such a case work loose, and under modern heavy traffic conditions, rapid and complete disintegration of the surface would follow.

Crushing Strength. Cylinders 1 in. in diameter and 1 in. long (see Figs. 5 and 6) are subjected to compression at the rate of 5 tons per minute until complete crushing of the specimen is observed. The crushing strength is usually reported in lb. per square inch. Results are sometimes given in tons per square feet and the approximate conversion factor is:

$$\text{Tons per square foot} \times 15{\cdot}555 = \text{lb. per square inch.}$$

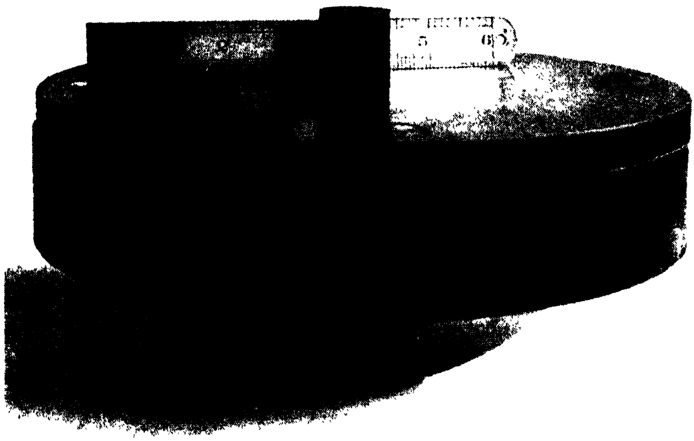

Fig. 5.—Stone Cylinder before Crushing Test.

Fig. 6.—Stone Cylinder after Crushing Test.

[*Facing p.* 20.

It must be appreciated that the normal weight of traffic exerts a considerably lower pressure than is needed to crush a specimen of even weak stone according to this test. On the other hand, cases of exceptionally heavy loads must be borne in mind, and in order to have a safety margin, it is unwise to use a stone with a crushing strength lower than 20,000 lb. per square inch for the wearing surface of roads carrying heavy traffic. Lower crushing strengths would be acceptable for base-coat work and for roads carrying lighter traffic, but it does not appear desirable to go below a minimum figure of 17,000 lb. per square inch.

In this respect, it must be noted that British Standard Specifications Nos. 594 [1] and 595 [2], dealing with rolled asphalt, specify a crushing strength minimum of only 14,000 lb. per square inch for " calcareous rock " (limestone). This figure is unnecessarily and dangerously low, and the writer understands that it was only agreed under protest by the Limestone industry Members of the British Standards Institution appropriate Committee.

This type of road construction, which is often used where heavy traffic conditions prevail, necessitates a high minimum strength, and a figure of 20,000 lb. per square inch would not only give a better indication of the quality of an average limestone but would also be a safer standard.

As mentioned in Chapter I, the British Limestone (Roadstone) Federation has recently carried out crushing tests on their members' stone in their research laboratory and the average crushing strength of 30 different samples was 27,800 lb. per square inch.

These samples came from all parts of the country and can be taken as representative of the better quality limestones quarried for roadstone purposes by manufacturers of repute.

A more recent test on a sample of Carboniferous mountain limestone gave the exceptionally high reading of 55,000 lb. per square inch, the average of four tests on the same deposit being 51,000 lb. per square inch.

Various references and recommendations concerning the use of " hard stone " have been made by Government Departments, and are incorporated in certain specifications. Without disputing the fact that hardness is a necessary characteristic of a road-stone,

[1] British Standard Specification No. 594 for Rolled Asphalt, Fluxed Lake Bitumen, and Asphaltic Bitumen, Hot Process. (British Standards Institution.)

[2] British Standard Specification No. 595 for Rolled Asphalt, Fluxed Natural Asphalt and Asphaltic Bitumen, Hot Process. (British Standards Institution.)

providing the necessary good adhesion properties are available, such a recommendation is very misleading, pending a definition of the term " hard stone ".

Some authorities have, with little justification, assumed that this term applies to igneous rocks only, and that if hard stone is specified, limestone is automatically excluded.

In view of the results just quoted, it can be definitely stated that good-quality limestone can be correctly included in the vague and unhappy description " hard stone ".

Impact Test. A standard hammer (weight 2 Kg.) is allowed to fall upon a plunger placed upon the top of a cylinder prepared from the stone, measuring 1 in. diameter by 1 in. high. The first fall of the hammer is from a height of 1 cm., and this is increased by 1 cm. for each succeeding blow until fracture of the test specimen occurs. The fall in centimetres of the hammer for the blow causing failure is reported as the measure of the toughness of the test specimen.

This test is of very little value under present traffic and road-surfacing conditions, with the exception of roads carrying a preponderance of horse traffic.

Water Absorption. This test was of considerable importance in the days of waterbound surfacings, but also has a certain significance under present road-surfacing technique, for the following two reasons:

(1) Chippings used for surface dressing which have a high water-absorption value are likely to give poor results and scale from the binder if applied shortly after heavy rain, as they will be wetter than less porous chippings, and will take longer to dry out.

(2) Stone used for the manufacture of bituminous material must be dried before coating with binder, and it is easier to dry a stone with a low water-absorption value than very absorbent stones. Drying temperatures can be kept lower and there is consequently less danger of the material and binder being " burnt ".

Average good quality limestones show low water-absorption values, and a recent series of tests on Carboniferous mountain limestones gave the excellent average of only 0·38 lb. per cubic foot or 0·22 per cent. by weight.

Specific Gravity. This test is of value to the engineer as it has a direct bearing on the cost of any form of construction.

The higher the gravity, the lower is the yardage covered by a ton of road-surfacing material or chippings, and a heavy concrete aggregate will produce a smaller volume of concrete than a light one.

The specific gravity of a hard limestone usually varies from 2·65 to 2·75. A recent average of 30 different samples of hard limestone gave a reading of 2·69. This compares very favourably with other stone aggregates normally used for road construction, some having a specific gravity as high as 3·00.

An apparently small difference in gravity can mean a big difference in the amount of materials used.

To summarise, good-quality Carboniferous mountain limestone fulfils all the physical requirements of a modern road aggregate. In addition, the excellent adhesion value of limestone is a great asset in the case of tar or bituminous surfacing materials and in the case of surface dressing.

CHAPTER III

ESSENTIAL CHARACTERISTICS OF A ROAD STONE

Safety—Durability—Economy—Specification for Road Stone

THE three main requirements of a road, i.e., Safety, Durability, and Economy, must be considered logically as a whole. It is, for example, useless to construct a road with an exceptionally good safety factor if the surface rapidly deteriorates or is exceedingly expensive to construct or maintain. Conversely, a road-surfacing material possessing an exceptionally long life is useless if, after a few years, it is to become dangerous to traffic.

The characteristics of a road surface depend on many variable factors, among which can be included—type of aggregate used, grading and type and amount of binder.

The choice of the right aggregate, although not the only factor, is of great importance, and the choice should be made so that all the previously mentioned essentials are maintained without any one feature receiving undue prominence at the expense of another.

Good-quality hard limestone fulfils these requirements in every way, and this has been adequately proved in actual practice and under closely controlled trial conditions.

SAFETY

The ideal road surface should be safe to all types of traffic which use it, i.e., motor vehicles, cyclists, horse-drawn vehicles, hunters, pedestrians, etc.

It is doubtful if any one surface will satisfy all these road-users.

The motorist demands a rugous finish which the cyclist and horse-owner dislike. The cyclist and pedestrian want a smooth " sandpaper " finish which the motorist, sometimes unjustly, condemns because it looks slippery. The horse-owner will complain of any surface on which his horse slips. Best results for horse traffic appear to be obtained with relatively smooth surfaces which, by reason of their fine-grained texture, give a sufficient number of points of contact to ensure enough grip between the shoe and the road surface to prevent slipping. The type of aggregate used in such a surface appears to be immaterial, as granite, gravel, limestone, or slag aggregates have all produced complaints from horse-owners. It must be noted in this respect that hunters appear to be more prone to slipping than farm horses or military mounts. Whether the condition of the horses' shoes, the standard of riding, or the influence which members of hunts have on local highways committees are responsible, is difficult to assess.

It is doubtful if the problem of providing a road surface which will satisfy all these users can be solved by any one surfacing material, and it would appear only logical to concentrate on a road surface which is safe to the majority of the users, i.e., motorists, and reasonably safe to other users. The full requirements of cyclists could be met by either providing separate cycle-tracks or, where the traffic does not justify this step, a width of about 3 ft. at the sides of the roads could be finished with a relatively smooth surface. The provision of grass tracks along the verges of the road would satisfy the requirements of horses.

Slipperiness. The possible factors which can cause a road to become slippery are many, including—type of aggregate, grading, type and amount of binder and surface texture, which latter is of ultimate importance and is largely governed by the grading and type and amount of binder. The influence of the type of aggregate used primarily affects the durability and economy of the road surface.

There has been considerable propaganda against limestone, it being claimed that limestone surfaces tended to powder and form a slurry which resulted in the formation of a slippery film on the surface of the road. This is definitely unjustified, and tests reported by the Road Research Board [1] disprove this statement.

In this test the coefficients of friction between a rotating rubber wheel and a test surface using different slurries between the surfaces, were carried out. Clay pastes of varying viscosities all showed lower friction coefficients than pure water. A suspension of limestone filler (passing 200 mesh) containing the same weight of solid material per cubic centimetre as the clay pastes, on the other hand, *raised* the coefficient above that of water.

Many forms of road construction are available which are satisfactory and safe from the motorist point of view. They include—tar or bituminous macadam, rolled asphalt, concrete and thin carpets.

The intensity of the traffic, type of foundations available, and ultimate cost, would largely determine the type of construction employed.

[1] Report of Road Research Board, Year ended 31 March, 1936, p. 115. (H.M.S.O.)

Whichever type is chosen, good-quality limestone will give completely satisfactory results. Official tests have amply demonstrated this fact. The following are a few examples of the excellent results obtained with limestone as an aggregate for bituminous work:

Ten trial sections were laid by the Ministry of Transport on the Kirkham By-pass in 1934 and the aggregates used included granite, limestone, slag and grit. The average sideway force coefficient since laid, of the Limestone Section G (two-coat bituminous macadam), is reported in *Experimental Work on Roads* [1] to be 0·66. This is the second-best reading of any of the sections, which did not require repairs, the best readings of all being only slightly higher at 0·71. The lowest average reading obtained was on a granite concrete section (D) with an average reading of 0·46. The lowest reading since laying of Section G was 0·59, which compares with a reading of 0·30 for the previously mentioned concrete section. These results must be considered in their right perspective, and it is not intended to convey the impression that limestone is superior in this respect to granite. It should be noted that two different forms of construction are being compared. The inference can be drawn, however, that the form of construction or technique, and not necessarily the aggregate, is an important factor on the question of slipperiness. These results do show, however, that a hard limestone aggregate can provide a satisfactory non-skid road-surfacing material.

The safety of a road surface does not depend entirely on a high sideway force coefficient. Another important factor is the absence of excessive irregularities in the road surface. If the riding qualities of a road are poor, this can be a contributing factor to an accident. In this respect, the Limestone Section G shows up exceptionally well. The number of inches of surface irregularity per mile were measured in July, 1937,[2] and Section G showed only 117 in. Only one section (Section D) gave a better reading, with 112 in., but it must be noted that the form of construction used in this latter section was concrete. The worst reading was on Section C (single-coat asphalt, granite and sand aggregate) with a reading of 490 in.

The exceptionally small amount of surface irregularity of the Limestone Section G can be attributed to the natural physical and chemical properties of limestone. It is well known that limestone not only possesses a higher natural affinity towards bituminous binders, so that it is difficult to displace the binder, but the texture

[1] *Experimental Work on Roads.* Report for 1938–39, pp. 66–70. (H.M.S.O.)

[2] *Ibid.* Report for 1937–38, p. 84. (H.M.S.O.)

of limestone is such that binders are readily held on its surface and do not drain away.

Limestone also has a " stiffening " effect towards binders, which results in a considerable increase in viscosity after relatively short periods. These characteristics all help to withstand the pushing effect of modern traffic and explain why tar or bituminous limestone surfacings normally have an even running surface, and retain these characteristics throughout their lives.

The tests on Section G must be considered highly satisfactory, especially when it is considered that this section had been subject to traffic for five years without requiring any attention, and that the traffic at the last census was 14,490 tons per day. The latest report of the Experimental Work on Highways Technical Committee [1] states that Section G was the best, closely followed by Section K.

Further evidence is found in " Road Research Technical Paper No. 1," where it is reported that tests were carried out on three tarmacadam roads laid with different aggregates in a city street carrying 10,000 tons of traffic per day. The aggregates were good-quality hard limestone, granite, and slag.

The sideway force coefficient figures were found to vary within very small limits, and the conclusion is reached that, according to these tests, the stone used in the surfacing has only a slight effect on resistance to skidding.

Six sections of tarmacadam were laid by the British Road Tar Association [2] on the Blackrod By-pass in 1934 in co-operation with the County Surveyor of Lancashire. The aggregates used consisted of granite, limestone, and slag, laid to a consolidated thickness of $3\frac{1}{2}$ in. on pitching and concrete foundations. All the sections stood up to heavy traffic for three years, despite difficulties encountered at the contractors' plants and adverse conditions under which the materials were laid. At the end of that period the granite and slag sections were re-surfaced, but until the early part of 1938 the limestone sections remained untouched, and the conclusion is reached that the limestone sections proved to be the most successful.

Colour. The colour of a road is also interconnected with its safety. A black polished surface makes driving difficult at night, and under certain conditions, such as heavy rain, positively dangerous. A very light colour, such as is presented by an untreated concrete road, can be dazzling and trying to the eyes in bright

[1] *Experimental Work on Roads.* Report for 1938–39, p. 69. (H.M.S.O.)
[2] British Road Tar Association, Report for July, 1938.

sunshine. The question of visibility from the air has also to be considered.

The ideal would appear to be as light a colour as possible without giving rise to dazzle. The grey or blue colour of limestone admirably fulfils these requirements.

The Road Research Board [1] have carried out tests on the light-reflecting values of chippings, which place limestone very favourably, as will be seen from the following figures.

The results shown in Table 5 were obtained by actually measuring reflected light with a " grease-spot " photometer, and those shown in Table 6, were the average consensus of opinion of fifty-five different observers after direct observation of the chippings.

TABLE 5

Light-reflecting Values of Chippings

Sample	Dry	Wet	Colour
Matt white paper	100		White
I LIMESTONE	98	60	White
F LIMESTONE	85	46	Greyish white
K Quartzite	82	59	Greyish white
B Gravel	78	50	Yellow
L Gravel (cement coated)	75	45	Light grey
H Granite (cement coated)	75	38	Light grey
A Granite	72	50	Light grey
J Gravel	70	50	Light red
C Granite	66	37	Pink
E Shale	63	44	Dark red
D Granite	60	35	Brownish grey
G Basalt	45	31	Dark grey
Blackened matt surface	26		

These results were to a large extent confirmed by trials carried out in the County Borough of Sheffield on Hathersage Road (A.625), and the following facts emerge from the report of the Experimental Work on Highways (Technical) Committee.[2]

The work was carried out in July, 1935, with the object of investigating the relative merits of various types of chippings from the point of view of night driving. The average daily traffic carried

[1] Report of the Road Research Board, Year ended 31 March, 1937, pp. 44–8. (H.M.S.O.)

[2] *Experimental Work on Roads.* Report for 1937–38, pp. 149–51. (H.M.S.O.)

TABLE 6

Comparison of Chippings by Observers

Material	Average order of merit	
	Chippings dry	Chippings wet
LIMESTONE	1	1
Granite (cement coated)	2	7
Quartzite	3	2
LIMESTONE	4	4
Granite	5	6
Granite	6	11
Gravel (cement coated)	7	9
Gravel	8	3
Granite	9	10
Gravel	10	8
Shale	11	5
Basalt	12	12

Note.—Table 6 is a composite representation of the results reported in two separate tables in the original report.

Tables 5 and 6 are reproduced from the Report of the Road Research Board for year ended 31 March, 1937, by kind permission of the Controller of H.M. Stationery Office.

was 5,274 tons (1935) and the trial was divided into 12 sections, each 85 yds. long, as follows:

Main Features of Construction

Chippings

Section A. ⅝-in. pink Leicestershire granite.
Section B. ⅝-in red Doncaster shale.
Section C. 78 per cent. of ⅝-in. Staffordshire gravel and 22 per cent. of ⅝-in. red Sheffield shale.
Section D. ⅝-in. Staffordshire gravel.
Section E. ¾-in. Doncaster gravel.
Section F. ¾-in.–⅜-in. Kirk Ireton gravel.
Section G. ⅝-in. local sedimentary stone.
Section H. ¾-in. basalt.
Section J. ¾-in. cement-washed basalt.
Section K. 50 per cent. of ⅝-in. *limestone* and 50 per cent. of ¾-in. basalt.
Section L. 64 per cent. of ⅝-in. *limestone* and 36 per cent. of ⅝-in. Doncaster shale.
Section M. 73 per cent. of ⅝-in. Horton stone and 27 per cent. of ⅝-in. Sheffield shale. (Abandoned owing to widening and realignment of the road.)

Binder

A proprietary tar-bitumen binder (tar of 90 sec. viscosity with 10 per cent. bitumen) applied at 240°–255° F.

In 1937–38 the Committee [1] reported that the gravel sections

[1] *Experimental Work on Roads.* Report for 1937–38, p. 151. (H.M.S.O.)

were, as a group, the best from the point of view of conspicuity, the *limestone* sections coming next. The basalt sections were poor in this respect. The order of merit of the best sections, between which there was little to choose, was F, E, D, L, C, B, A.

From the point of view of durability, the gravel sections were less satisfactory than the others.

Taking both durability and colour into consideration, Sections A and L were in the Committee's opinion probably the best.

Skidding tests made during the year gave consistently good results, the lowest coefficient obtained on any section being 0·48.

To summarise: practical experience supported by official trials have conclusively proved that good-quality limestone aggregate provides a perfectly safe surface for all types of tar or bituminous surfacings.

The characteristics of limestone concrete roads are fully discussed in Chapter IX, but it can be stated now that hard limestone aggregate makes as safe concrete as any other aggregate. It must be noted that the safety of concrete roads, irrespective of the aggregate used, is still a very open question. Some authorities maintain that concrete surfaces are slippery and there is certainly evidence to support their views. On the other hand, there are sufficient concrete surfaces available which are absolutely safe to demonstrate that slipperiness is not an inherent fault of concrete. The whole problem of the safety of a concrete surface appears to depend almost entirely on the finish of the surface of the concrete. A badly proportioned mix, combined with excessive tamping or vibration, can easily lead to the formation of a skin of concrete mortar on the surface which can readily " face up " and become slippery.

Ministry of Transport tests [1] carried out in Lancashire on the Maghull diversion lend support to this argument, the following sideway force coefficients having been obtained in June, 1937, on three granite-concrete sections:

Section G (4 : 2 : 1 mix).	Sideway force coefficient,	0·25
Section H (3 : 1½ : 1 ").	" " "	0·14
Section J (3 : 1½ : 1 ").	" " "	0·16

It will be seen that the lean mix (G), although giving a very low reading, is better than the two fatter mixes. The corresponding readings for the blue limestone section (0·18) and gravel (0·21) are almost exactly the average of the highest and lowest granite reading. The wide choice of aggregates used with almost identical results

[1] *Experimental Work on Roads.* Report for 1937–38, pp. 36–8. (H.M.S.O.)

confirms that the surface finish of the concrete is of more importance than the nature of the aggregate in connection with the slipperiness of the surface.

On the question of strength, the Experimental Work on Highways (Technical Committee) [1] report that a comparison of Sections L, M, N, and O showed the blue limestone aggregate to have the greatest strength.

The aggregates of these sections were as follows:

(L) Limestone.
(M) Granite.
(N) Gritstone.
(O) Gravel.

One of the final conclusions [2] on these trials is to the effect that satisfactory results can be obtained with a variety of aggregates, including granite, limestone, and gravel.

The many advantages of limestone as an aggregate for concrete are fully discussed in Chapter IX.

DURABILITY

The durability of a road surface depends on a large number of factors, of which a full survey is outside the scope of this book. A brief summary is as follows:

Uncontrollable Factors :

(1) Nature and intensity of traffic.
(2) Weather conditions.

Controllable Factors :

(1) Strength and nature of foundations.
(2) Type of lateral support (kerb, grass verge, etc., or none).
(3) Bond between surface and foundations.
(4) Specification, which includes:
 (*a*) Nature of aggregate.
 (*b*) Grading.
 (*c*) Type, quality, and quantity of binder.
(5) Correct manufacture (quantities, temperatures, etc.).
(6) Correct laying and rolling.

[1] *Experimental Work on Roads.* Report for Year 1934, p. 25. (H.M.S.O.)
[2] *Ibid.* Report for 1937–38, p. 38. (H.M.S.O.)

Of these controllable factors, the question of foundations and lateral support depends greatly on the funds available, but it is usually false economy to save on these items. No surface will remain durable for long, if the foundations are not strong enough to carry the present and *anticipated* traffic. Lack of lateral support results in the unsightly and dangerous rotting and disintegration of the edges of the road, leaving these edges dangerous to cyclists and reducing the effective width of the road.

The necessity of a suitable " key " or bond between the road surface and the foundations is generally recognised, but often neglected. There is, on the other hand, no need to waste money and do more harm than good by flooding a rough water-bound finish (which probably gives sufficient " key " as it is) with emulsion prior to the application of a carpet coat.

Suitable specifications for all the various forms of limestone surfacings will be found in subsequent chapters, together with notes on the correct manufacture and laying; but the influence of the type of aggregate on the durability of a road surface is sufficiently important to warrant a more detailed discussion here.

The chief causes of failure of a bituminous road or surface dressing are as follows:

(1) Actual wear of the surface.
(2) " Fretting " or working loose of individual particles or pieces of stone. Once such " fretting " has started, complete disintegration can often follow very rapidly.
(3) " Fattening up " or " bleeding " of binder, resulting in a slippery road.
(4) Displacement of the surface under the weight of traffic leading to the formation of waves, corrugations, etc. These phenomena often follow (3).

The wear on a modern road under present traffic conditions is exceedingly small; so small, in fact, that it is very rare indeed for a road to have to be resurfaced because it has worn away.

In almost every case reinstatement due to fretting or disintegration, bleeding or waviness has to be undertaken before the effect of actual wear is sufficient to warrant a replacement on this account. As will be explained later, the deciding factor and cause of such failure is usually lack of adhesion.

Limestone has long been recognised as the safest material to use in conjunction with tar or bitumen, but apart from statements that limestone was " kinder " to such binders, it is only of recent years

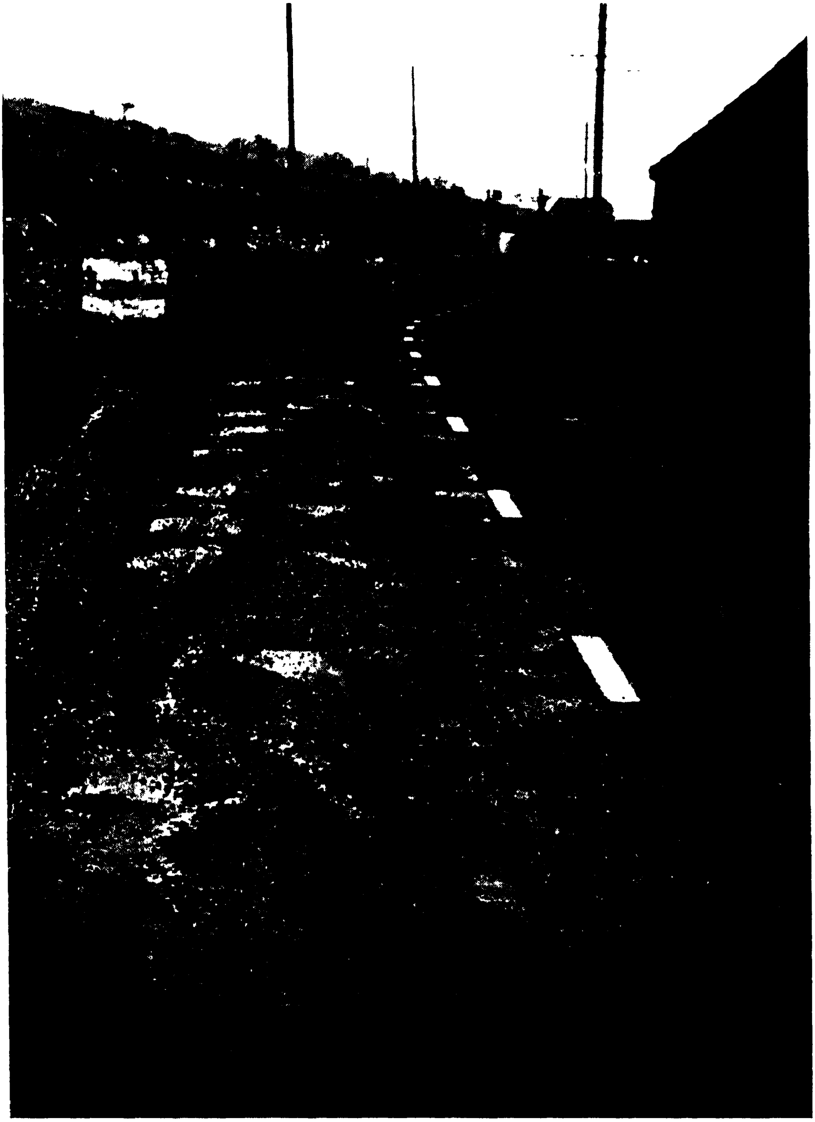

Fig. 7.—Stripping of Chippings from a Surface Dressing. (Adhesion value of chippings 1–3.)

[*Facing p.* 32.

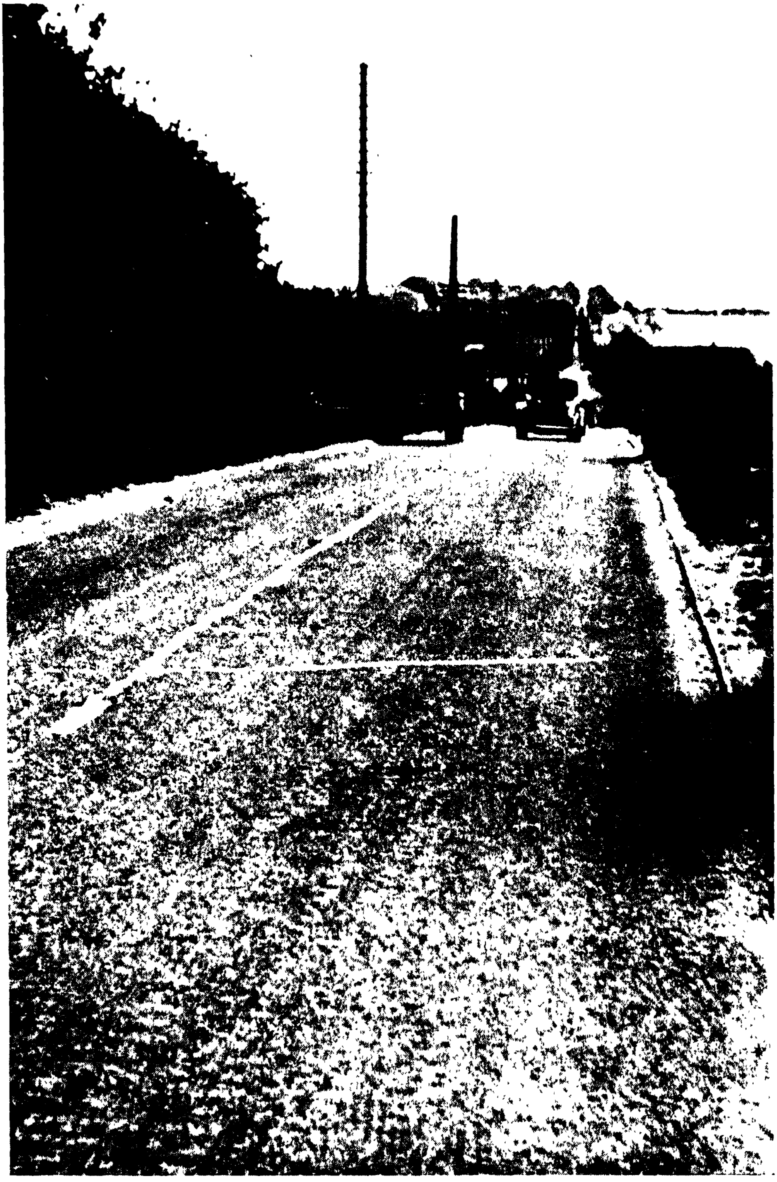

Fig. 8.—Hot Asphalt Surfacing (Mountain Limestone Aggregate), on A.33 at Shroner Hill, Kingsworthy. Laid 1935. Photographed 1945.

Reproduced by permission of Brigadier A. C. Hughes, C.B.E., T.D., B.Sc., M.Inst.C.E., County Surveyor, Hampshire County Council.

[*Facing p.* 33.

that modern research has converted this generalisation to a definite and scientific fact.

The salient point and the reason for the success of limestone is due to the exceptionally high degree of adhesion existing between this class of stone and bituminous binders. Details of the adhesion test and typical adhesion values of different road stones, which clearly showed the superiority of limestone in this respect, were given in Chapter II, but the importance of this characteristic is so great that it cannot be emphasised too much.

A definition of adhesion is " The property of holding particles together ", and this aptly describes what should be the case in a bituminous road mixture. If the adhesion between the stone and the binder used is poor and insufficient, there is nothing to stop two adjacent stones from separating. No matter how good the binder, and even if the binder films of two adjacent stones are firmly bonded, this will not prevent the actual stripping or displacement of the film of binder from the stones.

Once such displacement has taken place, further disintegration is rapid; the stones which have become loose in the structure of the surfacing readily loosen other stones by their movement under traffic.

Such disintegration will take place, whether the surfacing consists of a bituminous carpet or a surface dressing. The accompanying photograph (Fig. 7) shows a typical case of bad stripping of chippings from a surface dressing. The adhesion value of these chippings was 1–3. Not only the durability of the surface is affected by such stripping but the riding qualities of the road suffer as well. In this particular case, depressions of about half an inch have been formed where the chippings have stripped.

There is usually an increase in adhesion value with increased viscosity (or decreased penetration) of the binder used. This explains why certain aggregates which have proved very difficult to coat satisfactorily for cold or semi-hot bituminous work (where relatively low viscosities are used) have proved successful in hot asphalt work where a relatively hard asphalt is used.

It is not intended to imply by this that limestone is unsuited for such work. On the contrary, excellent results have been obtained. One county, for example, uses (and has used for a great number of years) hard limestone exclusively as the aggregate for the surfacing of their main roads with hot asphalt. Some have carried heavy traffic for twenty years. A typical example is illustrated in Fig. 8.

The value of limestone as an aggregate for hot rolled asphalt is borne out by W. J. Hadfield[1] who is of the opinion that limestone of suitable quality is a good aggregate for the wearing coat. W. J. Hadfield quotes out of many instances, two asphalt roads with a limestone wearing coat, each carrying about 9,000 tons per day, which have been down for seven and eight years respectively, and are in good condition.

Cases of roads constructed with limestone having exceptionally long lives are too numerous to mention, but the accompanying photograph (Fig. 9) showing a section of a limestone tar macadam road after twelve years' heavy traffic, is interesting. The traffic at the last pre-war census was over 10,000 tons per day. The dense structure of what must have been originally a mixture with a high void content and the absence of crushing or wear of the limestone, despite the heavy traffic it has carried, should be noted.

ECONOMY

The question of economy of a road-surfacing material must be considered not only from the aspect of initial cost, but must also take into account the probable life and expenditure on maintenance, so that an ultimate cost figure can be ascertained.

Limestone surfaces are economical for the following three reasons:

(1) Initial cost per ton as compared with most other stone aggregates is usually substantially lower.
(2) Lower gravity gives a greater spread (cf. Chapter II).
(3) Durability is equal to and often superior to other stone aggregates.

This brief exposition of the compliance of good-quality limestone with the three essential requirements of a road stone (safety, durability, and economy), explains why limestone was, and continues to be, one of the most extensively used stone aggregates in Great Britain.

SPECIFICATION FOR ROAD STONE

There are various British Standard Specifications dealing with broken stone and chippings, rolled asphalt, cold asphalt, tar macadam mixes, etc., but none of these specifications has attempted to define completely the essential characteristics of the aggregate.

The quality of stone within any geological classification can vary

[1] W. J. Hadfield, *Highways and Their Maintenance*, p. 131. "The Contractors' Record Ltd."

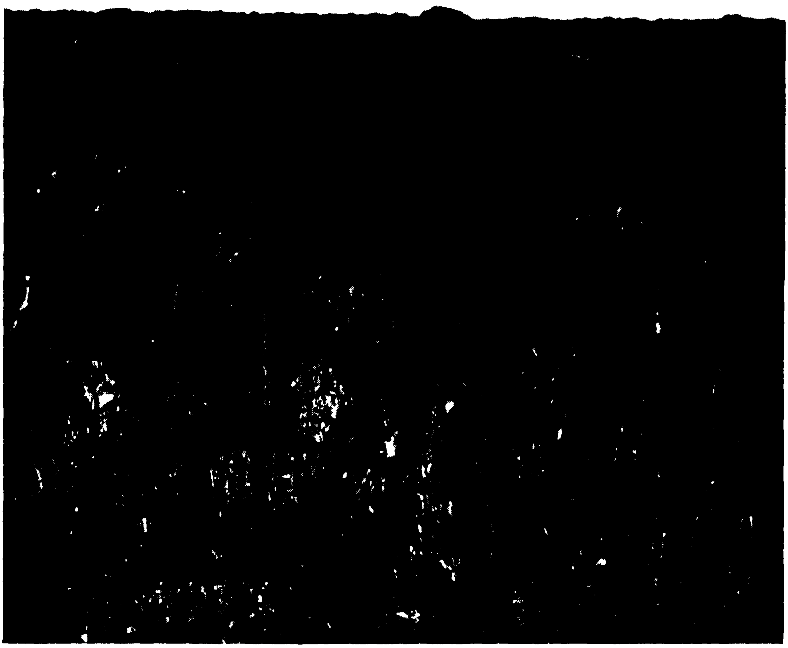

Fig. 9.—Section of Limestone Tarmacadam Surfacing after 12 Years' Heavy Traffic.

[*Facing p.* 34.

over very wide limits. The present system of accepting tenders on the basis of the lowest quotation irrespective of quality has resulted in many cases in the use of inferior materials, whether limestone, granite, basalt, or any other stone.

An up-to-date specification for all types of roadstones is long overdue. The limits suggested in Chapter II, with the addition of a minimum adhesion value of 3, could be a good working basis.

The writer understands that the British Limestone (Roadstone) Federation had this point well in mind insofar as limestones are concerned. Unfortunately, the war prevented the completion of necessary investigations, and it is hoped that they will be continued as soon as circumstances permit. The establishment of such standards should be of real benefit to the limestone and roadstone industries as it should lead to the elimination of poor-quality stone. The next logical step would appear to be the introduction of a special name for good-quality limestone. " Hard limestone " is one name that automatically suggests itself.

CHAPTER IV

UNCOATED LIMESTONE AS A ROAD AGGREGATE

Sizes of Broken Stone and Chippings—B.S. 63—Waterbound Macadam—Surface Dressing—Grouting—Silicate-bound Roads—Concrete Roads

Sizes of Broken Stone and Chippings. Increasing importance is being placed on the use of stone of correct size or of the right grading. This applies not only to coated stone and concrete aggregates, but also to clean stone, used as chippings for surface dressing or as an aggregate for waterbound construction or grouting.

Quarries have, generally speaking, kept pace with the higher quality demanded in this respect, and the days when quarries produced one size of " crusher-run " stone are now past, but there is still not complete standardisation amongst quarries to ensure that a consumer will receive the same size stone when ordering from two different quarries. This position has been largely created by the British Standard Specification for Sizes of Broken Stone and Chippings No. 63 : 1928, which was so impracticable in many aspects that it was rarely followed. This specification has now been thoroughly overhauled, and a revised specification, No. 63 : 1939, has been issued.

The essential features of the specification are reproduced in Table 7.

As the introduction of this specification entails a change in the designation of certain sizes of stone, a brief comparison of the old and new specifications should be helpful.

The testing and defining of the various stone sizes, according to the old specification, was carried out with round-aperture gauge-plates for all sizes from 3 in. to $1\frac{1}{2}$ in. inclusive. The $1\frac{1}{2}$-in. size, for example, was defined as material passing a $1\frac{1}{2}$-in. and retained on a 1-in. diameter ring. For the chippings sizes, from 1 in. down inclusive, square aperture holes were used for testing. As an example, 1-in. material was defined as material passing 1-in. square aperture and retained on $\frac{3}{4}$-in. square. In the accompanying diagram (Fig. 10) a 1-in. diameter circle is compared with a 1-in. square. From this it will be seen that stone passing through a 1-in. ring would normally be smaller than material passing through a 1-in. square. In other words, the lower defining limit of the

TABLE 7

Tolerances on Single-sided Material

Nominal size	Tolerance on oversize				Specified size			Tolerance on undersize			
	Passing B.S. sieve	Per cent.	Retained B.S. sieve	Per cent. max.	Passing B.S. sieve	Retained B.S. sieve	Per cent. min.	Passing B.S. sieve	Per cent. max.	Passing B.S. sieve	(Fines) Per cent. max.
2½ in.	3 in.	100	2½ in.	15	2½ in.	2 in.	65	1½ in.	5	7 mesh	2
2 "	2½ "	100	2 "	15	2 "	1½ "	70	1 "	5	7 "	2
1½ "	2 "	100	1½ "	15	1½ "	1 "	75	¾ "	5	7 "	2
1¼ "	1½ "	100	1¼ "	15	1¼ "	1 "	50	¾ "	5	7 "	2
1 "	1½ "	100	1 "	15	1 "	¾ "	55	½ "	7	7 "	2
¾ "	1 "	100	¾ "	15	¾ "	½ "	65	⅜ "	7	7 "	2
½ "	¾ "	100	½ "	15	½ "	⅜ "	55	¼ "	7	7 "	2
⅜ "	½ "	100	⅜ "	15	⅜ "	¼ "	60	$\frac{3}{16}$ "	10	7 "	2
¼ "	⅜ "	100	¼ "	15	¼ "	⅛ "	70	7 mesh	10	25 "	2
⅛ "	¼ "	100	⅛ "	15	⅛ "	10 mesh	65	14 mesh	10	25 "	2

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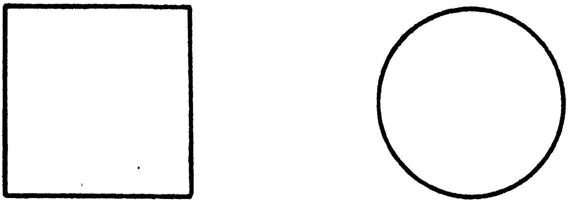

Fig. 10.—Comparison of Size of 1-in. Square and 1-in. Circle.

$1\frac{1}{2}$-in. stone was actually smaller than the upper defining limit of the next smaller size (1-in. stone). This impracticable state of affairs has been eliminated in the revised specification, by introducing square-aperture testing sieves for the whole range of materials and abolishing the round-aperture gauge-plates. It should be clearly understood that this new method of testing does not affect the shape of the screen plates fitted to quarry screens, and these can still consist of plates perforated with round holes. It is merely the method of testing and defining the sizes which has been altered.

The change from round to square holes for the testing of the larger sizes has necessarily altered the dimensions of these sizes. To take an example: the new $2\frac{1}{2}$-in. size is defined as passing $2\frac{1}{2}$-in.

TABLE 8

Original B.S. 63 : 1928			Approximate equivalent according to revision of B.S. 63 : 1939		
Nominal or specified size	Defining sieves		Nominal or specified size	Defining sieves	
	Pass	Retained		Pass	Retained
	ring	ring		square aperture	square aperture
3 in.	3 -in.	$2\frac{1}{2}$-in.	$2\frac{1}{2}$ in.	$2\frac{1}{2}$-in.	2 -in.
$2\frac{1}{2}$ "	$2\frac{1}{2}$ "	2 "	2 "	2 "	$1\frac{1}{2}$ "
2 "	2 "	$1\frac{1}{2}$ "	$1\frac{1}{2}$ "	$1\frac{1}{2}$ "	1 "
$1\frac{1}{2}$ "	$1\frac{1}{2}$ "	1 "	$1\frac{1}{4}$ "	$1\frac{1}{4}$ "	1 "
	square aperture	square aperture			
1 "	1 -in.	$\frac{3}{4}$-in.	1 "	1 "	$\frac{3}{4}$ "
$\frac{3}{4}$ "	$\frac{3}{4}$ "	$\frac{1}{2}$ "	$\frac{3}{4}$ "	$\frac{3}{4}$ "	$\frac{1}{2}$ "
$\frac{1}{2}$ "	$\frac{1}{2}$ "	$\frac{3}{8}$ "	$\frac{1}{2}$ "	$\frac{1}{2}$ "	$\frac{3}{8}$ "
$\frac{3}{8}$ "	$\frac{3}{8}$ "	$\frac{1}{4}$ "	$\frac{3}{8}$ "	$\frac{3}{8}$ "	$\frac{1}{4}$ "
$\frac{1}{4}$ "	$\frac{1}{4}$ "	$\frac{1}{8}$ "	$\frac{1}{4}$ "	$\frac{1}{4}$ "	$\frac{1}{8}$ "
$\frac{1}{8}$ "	$\frac{1}{8}$ "	$\frac{1}{16}$ "	$\frac{1}{8}$ "	$\frac{1}{8}$ "	10 mesh.*

* Approx. $\frac{1}{16}$ in.

square, retained on 2-in. square. These testing sieves will allow considerably larger stone to pass through than did the old 2½-in. and 2-in. round gauge-plates, and in actual fact, the new 2½-in. size is almost the same size as the old 3-in. The same remarks apply to the new 2-in., 1½-in., and 1¼-in. sizes, which are all approximately one size bigger than the old ones. Table 8 gives an *approximate* comparison of the two specifications.

It will be noted that there is no appreciable difference in the sizes from 1 in. down, inclusive.

Further anomalies which have been abolished by the new specification are the following:

(1) The old specification gave no indication of the grading of any permitted undersized stone. In the case of 1-in. chippings, for example, the specification merely said that all the sample should be capable of passing through a square hole of 1 in. side and at least 70 per cent. should be retained on a sieve having square holes of ¾-in. side. No mention was made of the composition of the 30 per cent. of material which was passing the ¾-in. sieve, and even if this consisted entirely of quarry dust, the chippings would still conform to the old specification. It will be seen by referring to the revised specification that the amounts of dust, etc., permissible, are clearly defined (cf. Table 7).

On the other hand, the percentages of " specified size " were, for some sizes, too severe, and these have in some cases been somewhat relaxed.

(2) No allowance for inevitable wearing of the quarry screens was made in the old specification, all sizes having to pass the larger defining sieve. In the revision, up to 15 per cent. of " oversize " is permissible.

Although the revision of B.S. 63 has been in force for five years, the changes which its adoption involves have not generally taken place. Consumers of stone, although sometimes specifying B.S. 63, still think in terms of the old round-aperture designation. On account of this the quarrying industry has on the whole been reluctant to take the radical step of re-naming the sizes it produces in accordance with the new specification, and, where necessary, of altering the aperture of screen plates. Stone which substantially passes a 1¼-in. square test sieve is still sold as " 1½-in.", although it should be described as " 1¼-in." under the new specification. Consumers are still ordering 1½-in. stone to conform to B.S. 63: 1939, fully expecting to obtain material similar in size to that which they obtained before the revision of B.S. 63, i.e., material substantially passing a 1½-in. ring. This is usually supplied, although material substantially passing a 2-in. ring should be supplied to meet the specification called for. Alternatively, often when the consumer orders correctly and specifies 1¼-in. stone, he is met with the answer that the supplier does not make that size, although in most cases material out of the so-called 1½-in. bin would conform in all respects to the requirements of the revised British Standard.

The time is now long overdue for either the producers or the consumers to enforce the new British Standard and to secure general adoption of the altered designations and methods of test.

Table 9 shows an approximate comparison of the two different designations:

TABLE 9

Designation according to old British Standard	Equivalent designation according to new British Standard
3 in.	2½ in.
2½ in. or 2¼ in.	2 in.
2 in.	1½ in.
1½ in.	1¼ in.
1 in. down	No change

Limestone has many uses as an uncoated aggregate, amongst which can be mentioned waterbound roads, chippings for surface dressing, aggregate for grouting work, concrete aggregate (see Chapter IX), silicate-bound roads, bottoming material, and foundations generally.

WATERBOUND ROADS

This form of construction is rapidly going out of favour, but where it is used, limestone is an excellent aggregate, especially if the road is to be subsequently surface-dressed, when the good adhesion properties of the limestone aggregate will assist in holding the film of binder. A typical specification is as follows:

SPECIFICATION No. 1

Waterbound Macadam on Existing Foundation

Preparatory Work. Any weak spots in the existing road should be cut out and made good.

Thickness. The consolidated thickness after rolling should be from 2 to 3 in., according to traffic requirements. If it is desired that the new coating should have a greater thickness than 3 in., the stone should be applied in two coats, rolled separately.

Cross Fall. The finished surface should have a cross fall of not more than 1 in 24, or ½ in. to the foot.

Size of Stone. A single size of stone will not give such good results as a medium-graded material. The following grading will give good results for a 3-in. consolidated surface:

Passing 2 -in. square mesh.	Retained on 1½-in. square mesh:	60 per cent.
" 1½ " " "	" " ¾ " " ":	30 " "
" ¾ " " "	" " ½ " " ":	10 " "

Spreading. Stone should be spread by experienced men. Stone should not be tipped on the road close to the point of spreading, but a few yards away so that the material is barrowed to the site, and an opportunity is afforded of re-mixing the material to counteract any segregation which has occurred during transit.

Spreading Capacity. One ton of limestone of the above grading will cover approximately 9 sq. yds. at a consolidated thickness of 3 in.

Rolling. This should be carried out by a roller weighing about 10 tons. Macadam should be consolidated by beginning work on the sides and working towards the centre. No water or binder should be applied until dry rolling has been carried out to a sufficient extent to afford a smooth, hard surface with the stones well knit together, showing their faces on the surface. Rollers should never stand up on new work for any length of time.

Binding. A mixture of limestone chippings and dust makes an excellent binding material, or failing this, a suitable pit sand or gravel can be used. The binding material should be spread, watered, and swept over the surface during the final rolling operation. It should be worked from the channels towards the centre so as to fill the interstices between the stones. Care should be taken not to use more binding material or water than is absolutely essential to ensure proper consolidation. One ton of binder material will treat approximately 40 sq. yds. of road.

Surface Dressing

General Principles. (1) Binders which are applied hot, whether tar or bitumen, will not adhere properly to a damp surface, and unless emulsions are used, good results will only be obtained during dry weather. Best results will be obtained if work is carried out during warm weather, when the road itself is warm, as the surface dressing compound can then be applied more readily and will adhere much better to the surface which is being treated.

(2) The surface to be treated must be absolutely clean and free from dirt, dust, or foreign matter.

Type of Binder. There are a great number of surface-dressing compounds on the market, and it is impossible to describe them all. Generally speaking, they can be divided into four classes:

Hot Application.

(1) Creosote fluxed asphaltic bitumens.
(2) Kerosene fluxed asphaltic bitumens.
(3) Tars.
(4) Straight-run soft bitumens.

Cold Application.

(5) Emulsions (tar and asphaltic bitumen).

Tar and emulsions have been standardised by the British Standards Institution. The appropriate numbers are 76 : 1943, 434 : 1935 and 618 : 1935.

According to B.S. 76, a type A tar should be used, and the viscosity ranges are given in Table 10.

TABLE 10

Tar Viscosities for Surface Dressing

	Type	Viscosity (expressed as seconds at particular temperatures, and as E.V.T.)	
		Winter	Summer
Surface Dressing: (*a*) For general purposes	A	20–40 at 30° C. E.V.T. 24–28° C.	80–120 at 30° C. E.V.T. 33–35° C.
(*b*) Waterbound macadam (first dressing)	A	10–20 at 30° C. E.V.T. 20–24° C.	20–30 at 30° C. E.V.T. 24–27° C.

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Note. Whilst for most purposes tar of the above-mentioned viscosities should be satisfactory for use in cold and warm weather respectively, it may be advisable to increase the viscosity up to 80 seconds at 35° C. (E.V.T. 38° C.) for the treatment during hot weather of roads carrying very heavy traffic.

As far as the asphaltic bitumens are concerned, the creosote fluxed products will usually give better results, especially if the surface to be treated is slightly damp. On the other hand, caution must be exercised when using this type of material on roads draining directly into fishing streams.

It is difficult to differentiate between the relative values of tar and asphaltic bitumen, opinion being very divided on this point, but there is no doubt that both materials can give satisfactory results if correctly applied.

The advantages of the addition of asphaltic bitumen to tar is another debatable point. Some authorities maintain that no advantage is to be obtained by such an addition; others insist that a noticeable improvement is obtained by the addition of 10 or 20 per cent.

There is no doubt that there is an upper limit, and that the addition of too much bitumen to certain tars (high aromatic or horizontal retort tar) can lead to precipitation of "free carbon" and resinous bodies from the tar in the form of a sludge. The binding and other essential characteristics of the tar will, in such a case, be seriously affected.

Safe maximum limits would appear to be 10 per cent. in the case of high aromatic tars (horizontal retort) and 20 per cent. for low aromatic tars (vertical retort). The vertical retort type of tar benefits most from the addition of bitumen. Such tars, although it is claimed that they are more durable than the old horizontal tars, are usually of a very slow setting type. This can be a disadvantage under modern traffic conditions and the addition of bitumen will usually result in a more rapid set.

It is safer to buy tar ready blended with bitumen than to try and do this oneself. The tar refiner knows the type of crude he is distilling and the best way of incorporating the bitumen in his tar.

Some authorities consider that the quality of some asphaltic bitumens is improved by the addition of tar. The amount of tar added should not exceed 20 per cent. or the previously mentioned precipitation troubles may be encountered.

SPECIFICATION NO. 2

Surface Dressing: Hot Application

Chippings. Size of chipping employed usually varies from ¾ in. to ¼ in. The smaller size is chiefly used for the dressing of footpaths. As far as roads are concerned, the sizes most commonly employed for main-road work are ¾-in. and ½-in. chippings. ⅝-in. chippings, although not a British Standard size, are often specified. $\frac{3}{8}$-in. will be found to be suitable for the dressing of roads carrying light or medium traffic.

Spraying Temperatures.

(*a*) Creosote fluxed bitumens: 250°–280° F. (120°–140° C.)
(*b*) Kerosene fluxed bitumens: 220°–240° F. (105°–115° C.)
(*c*) Tar and tar compounds: 220°–260° F. (105°–125° C.)
(*d*) Straight-run soft bitumens: 350° F. (175° C.)

There are certain proprietary fluxed compounds on the market which necessitate spraying temperatures of 300° F., but the above figures can be taken as representative of general current practice. Any prolonged heating in the spraying machine should be avoided.

Spreading Capacities.

(*a*) *Surface Dressing Compound.* The quantity of binder required depends chiefly on:

(1) The degree of absorbency of the road surface (or its texture).
(2) The size of the chips to be held.

In view of (1) it is impossible and unwise to state a definite figure, but Table 11, which is reproduced by permission of the British Road Tar Association, will give an approximate indication of the quantities required:

(*b*) *Chippings.* The following approximate spreading capacities apply to limestone. Correspondingly lower spreads will be obtained with stones of higher specific gravity.

Size	Approximate spread in square yards per ton
$\frac{3}{4}$ in.	70– 80
$\frac{1}{2}$,,	90–100
$\frac{3}{8}$,,	110–120
$\frac{1}{4}$,,	140–170

Method of Application. It is very necessary that the road should be clean. No surface dressing compound will adhere to a dirty surface. The surface should be thoroughly swept (and scraped if necessary) before spraying. Animal droppings "kill" any binder and should be *thoroughly* removed. Pot-holes, weak spots, and irregularities should be made good with coated stone, preferably a few days before commencing spraying.

TABLE II

Spreading Capacity, etc., of Tar (Hot Application)

	Yards per gallon of tar	Viscosity (seconds) B.R.T.A. at 30° C.	Temperature of tar	Gauge of chippings
For ordinary surface dressing with:				
1. ½-in. chippings	6	80–100*	220° to 260° F.	½ in.
2. ¾-in. chippings	4½			¾ in.
3. For the first dressing of a tar macadam road	4½	80–100*	do.	½ in.
4. For the first dressing of a water-bound road	Max. 3	20–40	Approx. 220° F.	½ in.
5. For dressing a carpet				Not to exceed ½ in.
6. For first dressing of concrete or other hard surfaces	6–7	60	220° to 260° F.	3/16 in.–⅜ in.
7. For second dressing of concrete or other hard surfaces	6	80–100		½ in.
8. For first dressing of wood pavements	8	100	220° to 260° F.	¼ in.–⅜ in.
9. For second dressing of wood pavements	6	80–100*		½ in.

* If for any reason the work has to be done in cold weather, it is important that the tar should be applied at a slightly higher temperature than in warm weather. If the normal practice is to use a high-viscosity tar, say, 80–100 secs. (B.R.T.A. at 30° C.), an alternative is slightly to lower the viscosity.

The road must be dry when the dressing is applied.

The above precautions may mean a little extra expense, but the cost of cleaning the road is a small fraction of the total cost of the dressing.

The hot compound can be spread mechanically or by hand. Mechanical spreading from tank wagons is cheaper, but it requires great skill of the operators to ensure uniform distribution and correct distribution according to the texture of the road. Unless mechanical gritters are employed it is very difficult for hand spreaders to keep up with the machine. Excellent results can be obtained by hand spreading.

One man should be detailed to follow the tarring gang, with a brush to paint over any untreated or badly coated spots.

The application of chippings should be carried out immediately after the dressing compound has been applied. There are a variety of mechanical gritters available, or the work can, if preferred, be carried out by hand. It is again advisable to have a man following up the work with a broom to sweep away an excess of chippings or to make good any deficiency.

The chippings used should be free from dirt or dust, and dry.

The chippings should be rolled immediately after application, avoiding any tendency to over-roll.

Surface Dressing with Emulsions. Emulsions can be broadly divided into three groups, as follows:

(1) Normal type.
(2) Semi-stable type.
(3) Stable type.

Emulsions consist of a finely dispersed mixture of bitumen and water. The colour of such an emulsion before use is brown, but after exposure to the atmosphere or contact with stone or other materials, the water separates from the bitumen, leaving a black film of this material on the surface treated. The water which has been set free ultimately evaporates. When the emulsion starts to turn black, it is said to have " broken ", and the chief difference between the three above types lies in the difference between the rates of " break ".

The normal type is the quickest-breaking variety, and the stable type is exceedingly slow breaking. The break of the semi-stable type lies between the two.

It is important only to use the normal or unstable type of emulsion for surface dressing or grouting work. These are covered by B.S. 434 (Asphaltic Bitumen Road Emulsion) and B.S. 618 (Emulsions of Road Tar and of Road Tar–Asphaltic Bitumen Mixtures). They are usually supplied in two grades with bitumen contents of 62 per cent. or 55 per cent., the former being suitable for surface dressing.

SPECIFICATION No. 3

Surface Dressing: Cold Application (Emulsions)

Method of Application. Before application of the emulsions, the road must be swept clean with a hard broom. Particularly dirty places should be scraped clean and animal droppings thoroughly removed. Pot-holes, weak spots, and irregularities should be made good with coated stone or with chippings which have been mixed with emulsion.

Emulsions can be successfully used on damp surfaces, in fact, in some cases it is better for the surface to be damp, than completely dry. On the other hand, emulsions will not adhere to very wet surfaces, and should not be applied during or between heavy rainstorms.

Before emptying the container, roll this backwards and forwards so that the material is well shaken and mixed.

The emulsion can be spread by hand, by pouring on the road with spraying cans fitted with special baffles, and working into the surface with a soft broom or squeegee. It is important to spread the emulsion with a single movement only, the operator pulling the emulsion towards him.

Alternatively, there are a variety of mechanical sprayers on the market, some of which are fitted with means of warming the emulsion to facilitate spraying during cold weather.

Chippings. Generally speaking, emulsions will not hold such large chippings as hot surface-dressing compounds, and it is not advisable to use sizes larger than ½ in. Emulsions will hold limestone chippings better than most other stones owing to the good adhesion properties of limestone towards bitumen.

Spreading Capacities. As in the case of hot surface-dressing compounds, it is impossible to give an exact figure, but Table 12 indicates the correct balance between the rate of application of the emulsion and the size and quantity of the chippings:

TABLE 12

Spreading Capacities of Emulsions and Chippings for Surface Dressing

Rate of application of emulsion	Corresponding size and quantity of chippings
5–6 sq. yds. per gallon	¼–⅜ in., 120–150 sq. yds. per ton
4–5 ” ” ” ”	⅜–½ ” 100–120 ” ” ” ”
3–4 ” ” ” ”	½–¾ ” 80–100 ” ” ” ”

The above figures only apply to surfaces which have been previously treated with tar or bitumen, and for more open surfaces the quantity of emulsion shown above for a given size of chip should be slightly increased.

It is advisable to detail a man to follow the spraying and spreading gangs to make good any deficiencies of application of the emulsion or irregularities in the spreading of the chippings. After spreading, the chipping should be rolled with an 8- to 10-ton roller, avoiding over-rolling.

An interesting development of surface-dressing lies in the treatment of one half-width of the road with a chipping of a lighter or different colour from that on the other half, thus creating two distinctive traffic lanes and a definite line of demarcation down the centre of the road. An excellent example of this type of work is reproduced in Fig. 11.

Limestone chippings are extensively used for surface-dressing work with excellent results; examples are too numerous to mention, but there is no doubt that a hard Carboniferous mountain limestone is eminently satisfactory for this class of work under heavy traffic conditions.

Limestone as a Grouting Aggregate

Grouting work can be divided into two classes, as follows:

(*a*) Full grouting.
(*b*) Semi-grouting.

Both these methods can be used with either hot compounds or with cold applied emulsion.

The chief differences between the hot and cold methods of application are as follows:

(1) Hot application can only be carried out during dry periods.
(2) The binders used for hot application are considerably thicker than emulsions, and the chief difficulty in their use lies in getting sufficient penetration of the binder through the layer of the stone. On the other hand, emulsions, on account of their extreme fluidity, flow through the layer of the stone readily and precautions have to be taken to prevent undue drainage of the emulsion to the bottom of the stone layer.

Limestone will be found to be an excellent aggregate for all types of grouting.

Specification No. 4

Full and Semi-Grouting: Cold Process

The following notes are in accordance with B.S. 433: 1931—the necessary adjustments to the stone gauges to comply with B.S. 63: 1939 having been made:

General. The consolidated thickness can vary between 2 in. and 4 in. The aggregate used should be clean and free from excess dust or dirt.

Fig. 11.—Surface Dressing using different coloured chippings.

[*Facing p.* 48.

Grading.

(1) *For 2-in. Consolidated Thickness :*

Stone	passing	1¼-in.	square.	Retained	on	1	in.	square :	60	per	cent.
„	„	1	„	„	„	¾	„	„ :	30	„	„
„	„	¾	„	„	„	½	„	„ :	10	„	„

(2) *For 2½-in to 3-in. Consolidated Thickness :*

Stone	passing	1½-in.	square.	Retained	on	1¼-in.		square :	60	per	cent.
„	„	1¼	„	„	„	1	„	„ :	30	„	„
„	„	¾	„	„	„	½	„	„ :	10	„	„

If a thickness greater than 3 in. consolidated is required, the work should be done in two separate courses, the second course being laid immediately the penetration of the first course is complete. In this case, the surface voids of the base course are not to be filled in with chippings, after the first application of emulsion.

Spreading. Scarifying should be unnecessary, unless the surface is very uneven and the camber requires much correction. Any irregularities should be made good and weak spots repaired.

Spread fine bedding material such as ⅛-in. limestone dust about ½ in. thick. This will spread at about 50 sq. yds. per ton. If the surface has been scarified, this may be omitted.

Spread the graded aggregate to the required contour, and to give the consolidated thickness required, and roll until firm. Each load of aggregate should be spread outside the area upon which it is dumped and spreading should be carried out with shovels (not forks). If the surface is to open or does not consolidate properly, ¾-in. chippings can be spread as required and rolling continued until the whole is thoroughly consolidated.

In the case of semi-grouting add sufficient water to form a slurry with the fine material in the base, and roll until this slurry has been worked up through one-half the thickness of the stone layer.

When the preparation of the stone course is finished, apply the emulsion evenly at the following rates :

Thickness of course		Gallons of emulsion per square yard
2 in.	Full grout . . .	1 –1½
	Semi-grout . .	½–1
2½ in.	Full grout . . .	1¼–1¾
	Semi-grout . .	¾–1¼
3 in.	Full grout . . .	1½–2
	Semi-grout . .	1 –1½

Rack up with ½-in. chippings, using just sufficient to fill the surface voids. Roll thoroughly and repeat rolling the day after construction. The road can now be open to traffic, but should preferably be given a seal coat after two days' traffic as follows:

Seal Coat. Sweep the surface clean and apply emulsion at the rate of 3–5 sq. yds. per gallon if using ½-in. chippings, and 4–6 sq. yds. per gallon if using ⅜-in. chippings. These chippings should spread at 100 to 150 sq. yds. per ton for the ½-in. size, and 120 to 180 sq. yds. per ton for the ⅜-in. size.

The chippings should be rolled after spreading.

On heavily trafficked roads a second application at the rate of 4 sq. yds. per gallon may be necessary after two or three months.

SPECIFICATION NO. 5

Full Grouting: Hot Process

In this process, tar or asphaltic bitumen are applied hot to a layer of stone, but many difficulties can arise owing to climatic conditions. The hot binder is rapidly cooled after coming into contact with the road stone, so that large voids must be left if full penetration is to take place, as otherwise, the binder fails to reach the lowest point in the stone. The use of a high-viscosity binder is necessary to prevent distortion, which again, however, increases the difficulty of obtaining penetration. Failure can also occur if the stone is only slightly damp, and will certainly follow if the stone is wet.

A suitable specification consists of straight ungraded 2-in. stone spread loose about 4 in. After light rolling, the binder, which should be heated to the maximum temperature specified by the manufacturer, is poured on to the stone at the approximate rate of 1½ to 2 gallons per square yard.

British Standard No. 76: 1943 recommends tars of the following viscosities for grouting:

	Type	Viscosity (expressed as seconds at particular temperatures, and as E.V.T.)	
		Winter	Summer
Grouting . .	A	30–50 at 30° C. E.V.T. 27–30° C.	80–120 at 30° C. E.V.T. 33–35° C.

Reproduced from B.S. 76 by permission of the British Standards Institution, from whom official copies of the specification may be obtained, price 2*s*. post free.

Note.—For work carried out during hot weather, it may be advisable to increase the viscosity up to 80 seconds at 35° C. (E.V.T. 38° C.).

Dry ½-in. to ¾-in. chippings should then be spread and the surface thoroughly rolled until there is no further movement. The surface should then be swept clean and a sealing coat applied (see Specification No. 2).

Limestone possesses many advantages over other aggregates for grouting work; the texture of the stone is just right to hold a maximum amount of binder and the high adhesion values of limestone are an added advantage. Hard limestones of the Carboniferous mountain type should be used on roads carrying heavy traffic, but the softer limestones are quite suitable for lightly trafficked roads.

SILICATE-BOUND ROADS

This form of construction can be considered an improvement on the ordinary waterbound type of construction, whereby instead of using water as the binding agent, a solution of sodium silicate is used.

Sodium silicate reacts chemically with limestone, forming an insoluble calcium silicate, and this form of construction can only be successfully used with limestone aggregate, or with an aggregate containing a relatively high proportion of calcium carbonate. This form of construction is not very extensively employed.

CONCRETE ROADS

Limestone is an excellent aggregate for the construction of concrete roads, and possesses many advantages which are dealt with in Chapter IX.

CHAPTER V

TAR AND BITUMINOUS MACADAM

General Principles—Measurement of Binder and Aggregate—Grading—Interpretation of Specification—Quality—Viscosity and Penetration—Tar Binder B.S. 76—Bituminous Binders—Manufacture—Tarmacadam—Bituminous Macadam—Laying of Tar and Bituminous Macadam—Spreading Capacity

THE first tarmacadam manufactured in the country was probably made with limestone aggregate, and the history of limestone tarmacadam can be traced back definitely to 1876. There is, however, evidence to support claims that limestone tarmacadam was used for footpath surfacing as early as 1832.

This early tarmacadam was mixed by hand, the stone was spread and warmed on hot plates, crude tar added with watering-cans, and the whole turned over with shovels or forks. The material was allowed to mature before being used. Nevertheless, the standard of quality obtained was often excellent and was no doubt largely due to the use of an aggregate which possessed a high affinity for the binder, and which was easy to coat even under primitive conditions.

Modern methods and machines have greatly helped to improve the quality and uniformity of present-day coated stone, but even now it is difficult to standardise the manufacture of these materials completely, owing to the large number of variables, such as stone size, grading, dust content, temperatures, and quality of binder.

In view of these unavoidable " variables " it is of the utmost importance to use a reliable aggregate which has been proved in practice.

General Principles. The same basic principles apply whether tar or a fluxed bitumen is used as a binder for the manufacture of pre-mixed stone which is designed to be laid cold or only slightly warm. The resulting product is in the one case, tarmacadam, and in the other, bituminous macadam.

The aggregate should have a good adhesion value towards the binder used (cf. Chapter II). It should be clean and free from any adhering dirt or dust. It should be dry and at the correct temperature. This temperature is usually of the order of 100° F. or 38° C. (hand warm), and in wet weather care has to be taken to ensure that the stone is really dry without exceeding the ideal mixing temperature. A well-equipped coating unit should have adequate cooling-off bins between the drier and the mixer, so that the stone can be heated to a sufficiently high temperature to ensure thorough drying, but has time to cool down to mixing temperature before being coated.

Correct stone temperatures are of the utmost importance. If the stone is too hot, the more volatile fluxing oils of the binder will evaporate, which will have the effect of artificially increasing the viscosity of the binder used, so that the resulting product may lack in ductility or be very stiff and difficult to handle once the material has cooled. There is also a danger that excess temperature of the stone will cause some of the binder to drain away, giving a load which is partly deficient in binder and part of which contains an excess.

The temperature of the binder also needs careful control, but providing it is not kept in store at high temperatures for long periods, less danger is likely to arise from the use of binder which is too hot.

The temperature of the aggregate has a marked effect on the rate of set of the coated material, a slow set being obtained with cool aggregates, and higher temperatures will cause a more rapid set. Although some manufacturers have views to the contrary, there is no evidence that the rate of set is greatly influenced by the temperature of the binder. It must be remembered that the binder is only approximately 5 per cent. of the whole, and shortly after addition to the stone in the mixer the binder must acquire the same temperature as the stone, which is substantially lower. As an example, one manufacturer specifies a binder temperature of 200° F. for normal-setting materials and 240° F. for quick-setting materials, although the ideal stone temperature is given as 100° F. in both cases. As uniform temperatures of 105° F. in one case and 107° F. in the other case are rapidly attained, it is difficult to appreciate the practical effect of the 2° increase in mean temperature. On the other hand, if the aggregate temperature were increased from 100° to 125° F., the extra 25° F. would, during transit to the site, cause appreciable evaporation of lighter fluxing oils.

When the stone is at the correct mixing temperature, a known weighed or measured batch is introduced into a mechanical mixer of the rotating paddle type, after which a weighed or measured amount of binder is added. Mixing usually takes from 2 to 3 minutes, and when every particle of stone is completely coated with binder, the manufactured material is discharged either direct into the transport vehicle or into a storage bin.

Continuous-mixing plants are still occasionally used, but cannot be considered satisfactory. The proportioning of the aggregate and binder is difficult to control and very dependent on the skill of the operative. Segregation difficulties may also be experienced with graded materials.

Measurement of Binder and Aggregate. Measurement of aggregate by volume is not so reliable as measurement by weight, for the following reasons:

(1) The number of stone particles in a given volume will vary appreciably according to the grading. A well-graded aggregate will contain less voids and more stone than, say, a single-sized stone. The surface area of stone to be covered with binder would thus be proportionately greater in the case of the denser gradings. Some variation in grading is inevitable, even in the best controlled plant, and if a certain grading is being coated with binder and the stone is measured by volume, to which a constant amount of binder is added, any batches which are more closely graded than standard will contain a deficiency of binder, whilst any fluctuation in grading tending to give a more open mix will carry an excess of binder.

To make this point clear, let us assume that the specification for a certain grading is 10 gallons per ton of stone, that batching by volume is used, and that the volume of a batch of stone is 1 cube yard. Let us further assume that for this particular grading, 1 cube yard weighs a ton, so that in order to comply with the specification requirements, 10 gallons of binder would be added per batch. If the grading of a batch is finer than specification, this will have the effect of decreasing the void content of the grading with a corresponding increase in weight. If the weight of such a fine batch (1 cu. yd.) is assumed to be 22 cwt. and the standard amount of binder is added (10 gallons per batch), then $\frac{10 \times 20}{22}$ or 9·1 gallons of binder per ton of stone will have been used. In other words, it will be seen that, although the grading is finer than specification and should carry more binder, a deficit of 0·9 gallons of binder per ton is present.

It is also difficult when measuring by volume to strike off accurately, each time, a constant volume of stone.

Measurement by weight largely overcomes these difficulties, providing an aggregate of constant specific gravity is used. This applies to limestone and other natural rocks, but not usually to slag, where the specific gravity can vary considerably and measurement by volume is usually employed. In addition, the texture of this material often varies so much that the correct amount of binder to be used fluctuates from batch to batch and is usually left to the discretion of the plant attendant.

Measurement of binders by volume can also lead to serious error, particularly if the binder froths. In such cases it is difficult to see the true level of the binder and froth is often measured as binder, leading to serious deficiencies. Any accumulation of foreign matter in the measuring pot will tend to give short measure. If this occurs when measuring by weight, an adjustment of the tare weight automatically compensates this error.

If measurement of binders by weight is used, a conversion from gallons to pounds is often necessary. The weight of a given volume of binder at a definite temperature can be calculated from the formula.

$$W = V \times \text{sp. gr.} \times 10 \text{ or, conversely,}$$
$$V = W \div (\text{sp. gr.} \times 10)$$

where W = Weight of binder in lb.
V = Volume in gallons at $T°$ C.
sp. gr. = Specific gravity at above temperature ($T°$ C.).

Binders are bought by the ton or by the gallon, but in either case the purchase is usually effected on a cold basis, i.e., 59° F. (15° C.). The binder is, however, used hot at temperatures from 170° to 270° F. (77° to 132° C.) which involves an increase in volume.

If the specific gravity at a given temperature is known, the gravity at other temperatures can be approximately calculated by using the appropriate constant shown in Table 13, multiplied by the difference in temperature of the two gravity readings. If the unknown gravity is at a lower temperature than the known, the correction is to be added, and if the unknown gravity is at a higher temperature, the correction is to be subtracted.

TABLE 13

Specific Gravity Correction per Degree Fahrenheit

Coal tar	0·00036
Asphaltic bitumens (liquid)	0·00030
Fuel oils (heavy)	0·00034
Diesel oils	0·00035
Kerosene	0·00040
Creosote	0·00042

For example, a consignment of tar is purchased on a basis of a specific gravity of 1·220 at 59° F. and the tar is used at 200° F. The difference in temperature is thus 141° F. This figure multiplied by the correction factor for tar of 0·00036 gives a correction of 0·05076 or 0·051 to the third decimal place. This correction subtracted from the gravity at 59° F. (1·220) gives the corrected gravity at mixing temperature of 200° F. as 1·169. This corresponds to an increase in volume of one-half a gallon on 11 gallons.

The following Table (No. 14) should prove useful to works which weigh their binder:

TABLE 14

Weight of a Gallon of Binder at Different Temperatures

Material	Sp. gr. at 59° F. (average value)	Weight of 1 gall. at 59° F. lb.	Approximate working temperature	Weight of 1 gall. at working temperature lb.
Horizontal tar . . .	1·220	12·2	200° F.	11·69
Vertical tar	1·170	11·7	170° F.	11·30
Creosote fluxed bitumen	1·020	10·2	240° F.	9·66
Kerosene fluxed bitumen	0·995	9·95	200° F.	9·53

Grading. The theoretical requirements of a good grading are governed by the maximum size of stone required and the percentage of voids desired in the mixed material. The top size of the stone is usually determined by the surfacing thickness required. As a general rule, material should be laid to a consolidated thickness not exceeding 1½ times maximum stone size. In other words, a 1-in. graded material should be laid about 1½-in. thick (consolidated).

The voids in tarmacadam are dependent on the proportions of the various bin sizes used, but it should be noted that cold-lay materials must contain an appreciable percentage of voids. These can only be eliminated by adding a high percentage of dust, and if considerable quantities of dust are added, it will be difficult or impossible to lay the material cold or even warm, and products resembling tar concrete or sheet asphalt will be obtained.

Up to 15 per cent. of quarry fines or 3 per cent. of filler dust can, however, be safely added. The addition of this dust will not only help reduce the voids, but also has the effect of increasing the viscosity of the binder used, and if a "fat" mix is being made, it will prevent the drainage of binder from the stone. The dust should be added preferably last, after the aggregate has been coated with binder.

The increase in viscosity, obtained with limestone dust, is quite substantial, with the added advantage that the increase is not immediate, but takes approximately twenty-four hours to attain its maximum value. In other words, the tarmacadam has usually been spread and rolled before the stiffening-up really takes place, but when this does occur, it is at the time when it is most needed, i.e., when traffic commences to use the newly laid surface.

The Limmer and Trinidad Lake Asphalt Co., Ltd.,[1] publish the following interesting figures, which clearly show the increase in viscosity of a tar to which 5 per cent. of limestone dust has been added, the increase reaching a maximum within 24 hours:

Hours after mixing	Tar alone	Tar plus 5 per cent. granite dust	Tar plus 5 per cent. limestone dust
Nil	36 seconds	44 seconds	44 seconds
3	36 "	44 "	47 "
6	36 "	44 "	48 "
24	36 "	44 "	50 "
48	36 "	44 "	50 "

Interpretation of Specification. The grading employed is very often specified by the purchaser either by reference to the relevant British Standard or in accordance with the purchaser's own specification. A common error is to assume that the grading figures given represent the percentages of stone to be used from the various bins. Sometimes it is not realised that it is impracticable to issue a specification giving the percentage of bin sizes to be used, on account of the fact that the size or grading of individual sizes from different quarries is bound to fluctuate even if they are within the limits of B.S. 63. It is accordingly only possible to specify the grading of the material after manufacture, and producers when studying a specification should bear this fact in mind and make the necessary calculation to ascertain the correct percentages to be used of each bin size.

To take a simple example, let us assume the case of a specification calling for the following requirements:

Passing 1½ in., retained on 1 in.:	60 per cent. ± 10 per cent.
" 1 " " " ¾ ":	35 " " ± 10 " "
" ¾ ":	5 " " ± 5 " "

Let us further assume that the manufacturers' 1½-in. and 1-in. materials have the following gradings which are well in accordance with British Standard requirements:

	Material from 1½-in. bin	Material from 1-in. bin	Material from ¾-in. bin
Passing 1½ in., retained on 1 in.	75 per cent.	—	—
" 1 " " " ¾ "	20 " "	80 per cent.	—
" ¾ "	5 " "	20 " "	100 per cent.

[1] The use of Trinidad lake bitumen in combination with tar for road construction.

If the manufacturer uses 60 per cent. from his 1½-in. bin, 35 per cent. from his 1-in. bin and 5 per cent. from his ¾-in. bin, the grading of the mixed product by calculation, will be:

Passing 1½ in., retained on 1 in.: 45 per cent.
" 1 " " " ¾ ": 40 " "
" ¾ " : 15 " "

This material is not in accordance with specification which calls for at least 50 per cent. of material passing 1½ in. retained on 1 in., and not more than 10 per cent. passing ¾ in.

By using bin percentages of 75 per cent. 1½ in. and 25 per cent. 1 in. material good agreement with the specification is obtained with a grading on calculation of:

Passing 1½ in., retained on 1 in.: 56¼ per cent.
" 1 " " " ¾ ": 35 " "
" ¾ " : 8¾ " "

This grading is well within the specification limits.

There are many methods of expressing a grading either in the form of a specification or as a report on a sieve test.

The differences lie in the method of expressing the percentages of each size and two methods are in general use:

(i) Percentage passing and retained between successive sieves.
(ii) Cumulative percentage passing each sieve.

The majority of producers and consumers of tar or bituminous macadam up to now employed the first method, which, although simple and straightforward, has considerable disadvantages and limitations from the point of view of the testing station or the Committee framing the specification. Table 15 compares the two methods of recording an identical grading:

TABLE 15

METHOD 1			METHOD 2	
Percentage passing			Cumulative percentage passing	
Sieve size		Percentage	Sieve size	Percentage passing
Passing	Retained			
—	1½ in.	0	—	—
1½ in.	1 "	42	1½ in.	100
1 "	¾ "	33	1 "	58
¾ "	¼ "	19	¾ "	25
¼ "	200 mesh	4	¼ "	6
200 mesh		2	200 mesh	2

Each method has its advantages and it is difficult to state which is the best. Method 2 is very extensively used in the concrete industry and Method 1 finds favour in the road industry. British Standard Specifications use both methods indiscriminately, but the writer understands that an attempt at standardisation in this respect will be made shortly.

A good general rule to determine the best proportion of stone is to divide the grading into three arbitrary zones, which, for convenience, can be called " Bigs ", " Middles ", and " Smalls ". In the case of a 1 in. graded carpeting material, the zones would be as follows :

Passing	1	in.,	retained on	$\frac{3}{4}$	in.	Bigs
"	$\frac{3}{4}$	"	" "	$\frac{1}{2}$	"	
"	$\frac{1}{2}$	"	" "	$\frac{3}{8}$	"	Middles
"	$\frac{3}{8}$	"	" "	$\frac{1}{4}$	"	Smalls
"	$\frac{1}{4}$	"	" "	$\frac{1}{8}$	"	

A well-balanced, reasonably voidless grading will be obtained if the percentage of " big " size is kept at 50 to 60 per cent., the " middles " at as low a figure as possible and not exceeding 25 per cent., and the " smalls " at 35 to 25 per cent.

While considering voids, it should be borne in mind that the high affinity of limestone towards binders gives considerably greater tolerance in the design of the grading than is possible with stones of lower adhesion values.

The amount of binder is usually specified as gallons per ton. It is important to differentiate between gallons per ton of stone and gallons per ton of coated material. Using a tar binder, 10 gallons per ton of stone are approximately equivalent to $9\frac{1}{2}$ gallons per ton of mixture. The difference increases with increased gallonage, and a sound specification should indicate which method of expression is intended. Expression of the binder content as a percentage by weight is probably more satisfactory, but is rarely used in the case of tarmacadam and similar materials.

Quality. Although the above-mentioned difficulties which have to be taken into account when manufacturing tar or bituminous macadam may not directly affect the consumer, it is hoped that they will be of value, if only to stress the point that the manufacture of such materials has made rapid strides during the last few years and has developed into a scientific manufacturing process where careful control is necessary throughout the mixing process. The pitfalls are many, and, unfortunately, good and bad tarmacadam or bituminous macadam are both black when delivered and look very much the same. The use of inferior coated stone could be almost entirely eliminated by purchasing these materials from reputable manufacturers. Alternatively, such materials should be ordered on the basis of a precise but practicable specification, and steps should be taken to test materials to ensure compliance with the specification. This would have the effect of eliminating cut-price materials, where the low price is only possible by sacrificing the quality and usually by reducing the binder content to the barest minimum necessary to ensure complete coating.

The present system of placing orders with the lowest tenderer, irrespective of the quality of the material, is not only basically unfair but false economy, as the following approximate calculation will show:

Job:	1½ in. single coat bituminous macadam.
Thickness:	2 in. consolidated.
Spreading capacity:	10 sq. yds. per ton.

A difference in tendered price of only 6*d.* per ton is, under present conditions, usually sufficient to determine the allocation of the contract. On the above spread this represents six-tenths of 1*d.* per square yard. The more expensive reliable material could be expected to have a life of at least three years before requiring surface dressing. Inferior material may commence to disintegrate within a few months of laying and need immediate surface dressing to avoid a complete failure. Apart from the cost of the dressing, in many cases, damage to the internal structure of the surface will have occurred before it is sealed off. It is highly probable that bad-quality material of this description will require surface dressing again at the end of three years, so that the original saving of 6*d.* a ton or 0·6*d.* per square yard results in a net loss of 5·4*d.* per square yard if we assume the cost of surface dressing to be 6*d.* per square yard.

Viscosity and Penetration.

(*a*) *Viscosity.* A brief description of the viscosity and penetration test may be of some value at this stage. The " viscosity " of a fluid is the scientific name for its property of offering resistance to motion. Common liquids have very different consistencies; water is a thin liquid and pours easily and does not offer much resistance to motion when, for instance, a cup of tea is stirred with a spoon. Golden syrup is a thick liquid which flows slowly and offers such

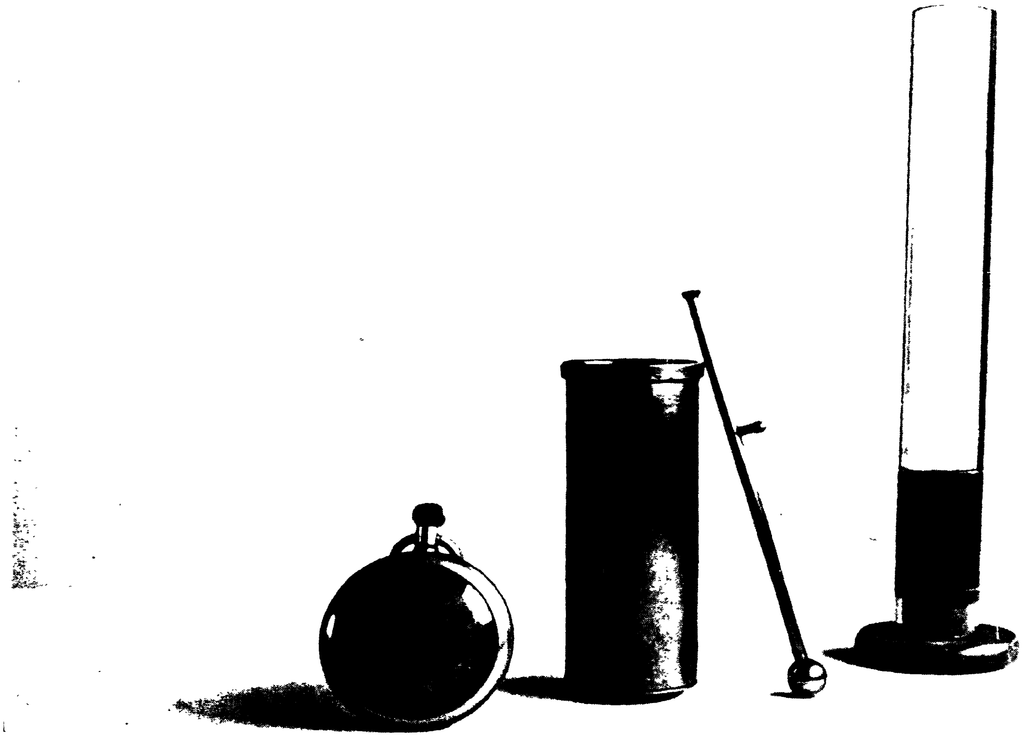


Fig. 12.—Standard Cup, Ball Valve and Measuring Cylinder, used in the B.R.T.A. Viscosity Test.

[*Facing p.* 60.

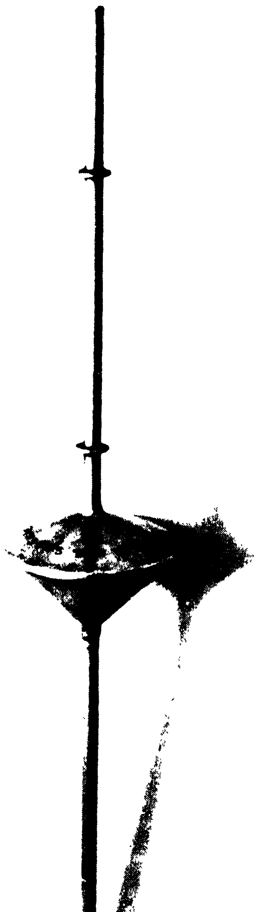


Fig. 13.—Hutchinson Tar-testing Spindle.

[Facing p. 61.

great resistance that it cannot be stirred quickly with a spoon. Syrup compared with water is said to have a higher viscosity, or to be more viscous.

The viscosity of tars and sometimes cut-back bitumens is measured by means of an instrument standardised by an association of manufacturers, the British Road Tar Association (B.R.T.A. viscometer). The cup shown (Fig. 12) has a hole 1 cm. in diameter in the bottom, which is closed by the valve (shown outside the cup) consisting of a ball on the end of a rod. Binder is poured in the cup till level with the peg fixed in the rod and cup and the contents are brought to a known constant temperature in a water-bath. The valve is taken out and the time taken with a stop-watch for 50 c.c. of the binder to flow into the glass measuring cylinder. The arrangement is a modification of Sir Boverton Redwood's lubricating-oil viscometer, which has, however, a much smaller orifice in the cup to deal with a thinner liquid.

Seconds are timed for the flow of binder through the orifice in the bottom of the cup. The thread of binder falls into the oil at the bottom of the glass measuring cylinder, which provides a surface to be read for timing between marks at 25 c.c. and 75 c.c. on the cylinder.

The test is inaccurate if the viscosity of the tar at the testing temperature exceeds 140 seconds.

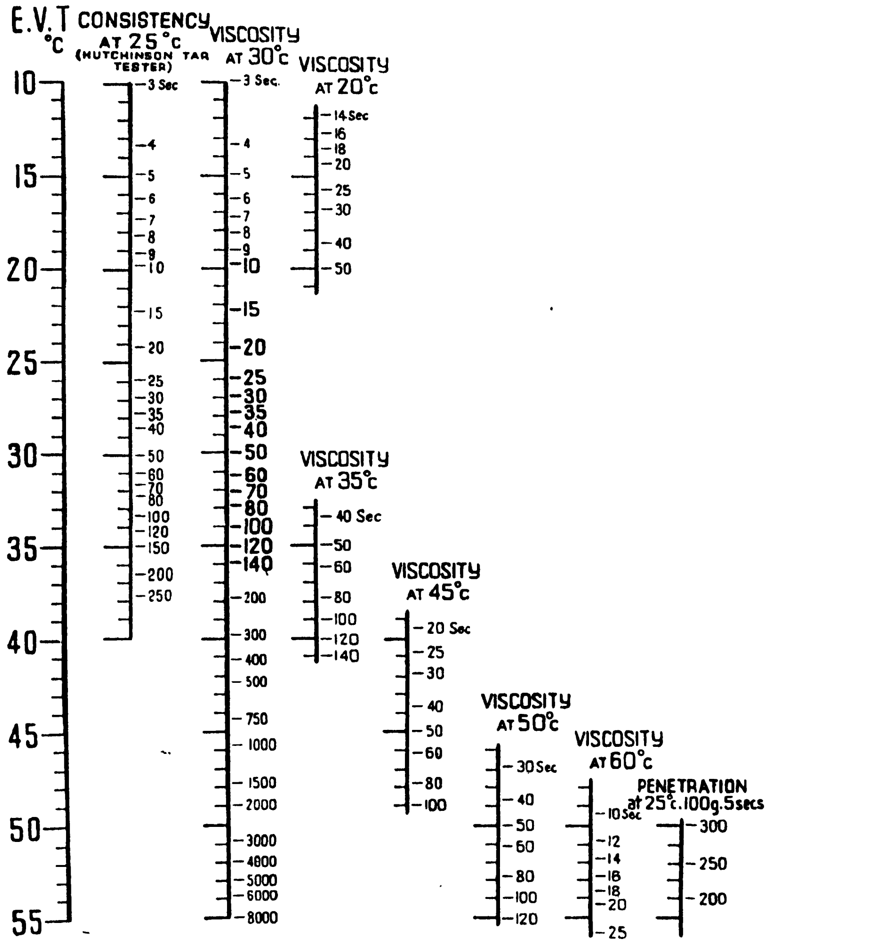
The temperature of the test is usually 30° C., but must be stated, as the same tar having a viscosity of 100 seconds at 30° C. would show only 45 seconds at 35° C. The tar becomes thinner on warming and will run through the orifice in a shorter time. Testing temperatures higher than 30° C. have to be employed when thick tars are tested. The Hutchinson tar tester is still used by the Bitumen Industry for the viscosity test of cut-backs. The sample of binder is brought to the correct testing temperature, usually 25° C., and the spindle (Fig. 13) introduced vertically. A stop-watch is started when the lower ring touches the surface of the binder and stopped when the second ring is in contact.

To avoid the confusion arising when the temperature of test or method of test is not given, the E.V.T. method of recording tar viscosities is coming into use. The equi-viscous temperature for a binder is the temperature at which the viscosity is 50 seconds B.R.T.A.

The use of this method enables the viscosity of any tar, no matter how fluid or viscous, to be expressed on a single scale. The higher the E.V.T. figure, the higher the viscosity.

Conversion from E.V.T. to the consistency at 25° C. (77° F.), using the Hutchinson tar tester, or the viscosity at different testing temperatures, using the B.R.T.A. viscometer, can be readily effected by means of the table reproduced below. This table cannot be used for the conversion of bitumen viscosities.

TABLE 16



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" *Note.*—The actual E.V.T. can be obtained only by the method given in 'Standard Methods', Serial No. R.T. 3–38 (p. 104).

Only viscosities within the limits 10–140 seconds may legitimately be determined on the Standard Tar Viscometer (see 'Standard Methods', p. 97). Figures outside these limits are given in the chart above for the purpose of rough comparison.

To obtain from a given E.V.T. the equivalent viscosity in seconds at one of the test temperatures shown, a straight-edge should be laid at right angles across the scales and the reading made on the appropriate viscosity scale. The chart also includes the approximate relationship between E.V.T. and the consistency at 25° C. as obtained with the Hutchinson Tar Tester as described on page 17 of the 1930 issue of B.S. 76."

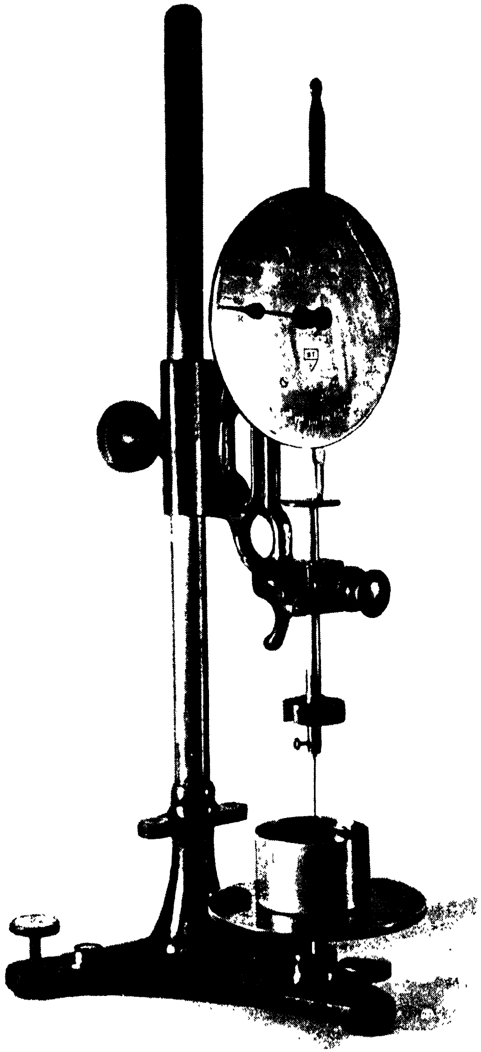

Fig. 14.—Penetrometer.

[*Facing p.* 62.

(*b*) *Penetration Test.* Asphalt has always been made with bitumens so thick that they would not flow through a viscometer orifice, and the penetration test became the standard method of finding their consistency. A needle is loaded to weigh 100 gms. and allowed to penetrate from the surface into a sample of bitumen. After 5 seconds the needle is clamped and its penetration into the bitumen is read in tenths of a millimetre. The higher the temperature of the bitumen under test the softer it will be and the greater the penetration. The usual temperature is 25° C. and the result is given as, say, " penetration 200 (25° C. 100 grs. 5 secs.) ".

It should be noted that the lower the penetration figure the harder the bitumen, which is the opposite to the viscosity figures in seconds time of flow. A penetrometer is reproduced under Fig. 14.

Tar Binder. B.S. 76. British Standard Specification No. 76 : 1930 was revised in 1943.

There are certain fundamental alterations in this revision which warrant some explanation.

The old specification divided road tars into three types based on viscosity, No. 1 tar covering the lowest viscosity range and being generally suitable for surface-dressing, No. 2 tar was of intermediate viscosity suitable for the manufacture of tarmacadam, and No. 3 tar was a very viscous tar suitable for the manufacture of semi-hot or hot-lay materials. The new specification also foresees three types, but these are based on road requirements and not viscosity. The differences are related to the rates of setting of the three types of tar on exposure as distinct from the immediate set, due to fall in temperature at the time of application.

Type A contains the highest proportion of lower boiling oils, and is designed for surface-dressing and for the manufacture of tarmacadam.

Type B contains a lower proportion of lower boiling oils, and is designed both for carpets and tarmacadam.

Type C is a special material, and in general is suitable for carpets containing a low proportion of fines. B.S. 76 : 1943 states that this tar is at present (presumably 1943) available in limited quantities only.

Changes have been made in the schedule of requirements, and the permissible variations in the proportions of oil, etc., are less. The proportions of phenol and naphthalene have been reduced and a new requirement has been included, as the softening point of the distillation residue or pitch is given. It is worthy of note that the effect of the nature of the crude tar used on the properties of the resulting road tar is not defined or specified. This is admittedly a very difficult problem to deal with in a British Standard, but the fact remains that a tar manufactured from vertical crude strictly in accordance with B.S. 76 : 1943 will not necessarily have the same characteristics as a similar tar made from a horizontal crude.

B.S. 76 : 1943 incorporates a table which is intended to serve as a guide in the choice of road tars suitable for various forms of road construction, but it is pointed out that it is in no sense obligatory or exhaustive. This table is reproduced below (Table 17).

It should be noted that the viscosities of tars used for the manufacture of tarmacadam specified in B.S. 802 : 1945 (cf. Table 22) differ somewhat from those specified in B.S. 76. In addition, the viscosities specified in B.S. 802 are now obligatory.

It will be seen from Table 17 that type A tar is recommended for all purposes, with the exception of carpets containing less than 30 per cent. aggregate passing a $\frac{1}{8}$-in. mesh sieve.

Table 17 gives considerable latitude on the question of the viscosity of the tar used for any particular type of material, particularly if the notes are considered as well. This confirms the fact that it is impossible to lay any hard and fast rules as to the viscosity of the tar to be used. This will depend on the type of limestone, size of aggregate to be coated, prevailing weather conditions, the time after manufacture when the material is to be used, special requirements of the customer and also the nature and the composition of the tar itself. As a general rule, the highest viscosity, compatible with ease of laying, should be used.

Reputable firms of tarmacadam manufacturers have a large fund of experience to draw from and the question of the most suitable viscosity is best left in their hands.

The types of material which have to be manufactured, and the requirements of customers vary so much, that to avoid stocking a great number of tars of different viscosities, it will be found advantageous to stock only two tars, one of the lowest viscosity which will be needed, and the other corresponding to the highest viscosity. By blending suitable percentages of these two tars, any intermediate viscosity can be obtained. Such blending can be carried out in the binder measuring trough, providing suitable means of agitation are available. Fig. 15 can be used for this purpose. To use this table, draw a line connecting the viscosity of the thin tar on the left-hand ordinate to the viscosity of the heavy tar on the right-hand ordinate. The percentage of low-viscosity tar required to give any intermediate

TABLE 17

	Type	Viscosity (expressed as seconds at particular temperatures, and as E.V.T.) Winter	Summer
1. *Surface Dressing.*			
a. For general purposes	A	20–40 at 30° C. E.V.T. 24–28° C.	80–120 at 30° C. E.V.T. 33–35° C.
b. Waterbound macadam (first dressing)	A	10–20 at 30° C. E.V.T. 20–24° C.	20–30 at 30° C. E.V.T. 24–27° C.
2. *Tarmacadam.*			
a. Base-coats . . . *b.* Wearing-coats to be surface dressed (within 12 months)	A	30–50 at 30° C. E.V.T. 27–30° C.	80–120 at 30° C. E.V.T. 33–35° C.
c. Hot process . . .	A	50–80 at 50° C. E.V.T. 50–53° C.	80–120 at 50° C. E.V.T. 53–55° C.
3. *Carpets.*			
a. Containing more than 30 per cent. aggregate passing a ⅛-in. mesh sieve	A	30–50 at 30° C. E.V.T. 27–30° C.	80–120 at 30° C. E.V.T. 33–35° C.
b. Containing 20 to 30 per cent. aggregate passing a ⅛-in. mesh sieve	B*	45–60 at 35° C. E.V.T. 34–36° C.	80–120 at 35° C. E.V.T. 38–40° C.
	or C	45–60 at 35° C. E.V.T. 34–36° C.	80–120 at 35° C. E.V.T. 38–40° C.
4. *Grouting*	A	30–50 at 30° C. E.V.T. 27–30° C.	80–120 at 30° C. E.V.T. 33–35° C.

" NOTES.—*Surface Dressing.* Whilst for most purposes tar of the above-mentioned viscosities should be satisfactory for use in cold and warm weather respectively, it may be advisable to increase the viscosity up to 80 seconds at 35° C. (E.V.T. 38° C.), for the treatment during hot weather of roads carrying very heavy traffic.

Tarmacadam and Carpets. In most cases tars of the above-mentioned viscosities should be satisfactory for use in cold and warm weather; it may be advisable to increase the viscosity up to 80 seconds at 35° C. (E.V.T. 38° C.), for tarmacadam base-coats and wearing-coats laid during hot weather and to reduce the lower viscosity to 20° C. E.V.T. (approx. 10 seconds at 30° C.) in very cold weather. For medium and open-textured carpets laid during hot weather on roads carrying very heavy traffic, it may be advisable to increase the viscosity up to 26 seconds at 45° C. (E.V.T. 41° C.).

Grouting. For work carried out during hot weather, it may be advisable to increase the viscosity up to 80 seconds at 35° C. (E.V.T. 38° C.).

* The tar producer and the aggregate supplier should be consulted on the selection of the type of tar to be used, taking as a principle that with carpets containing the lower proportion of fines the use of type C tar is preferable."

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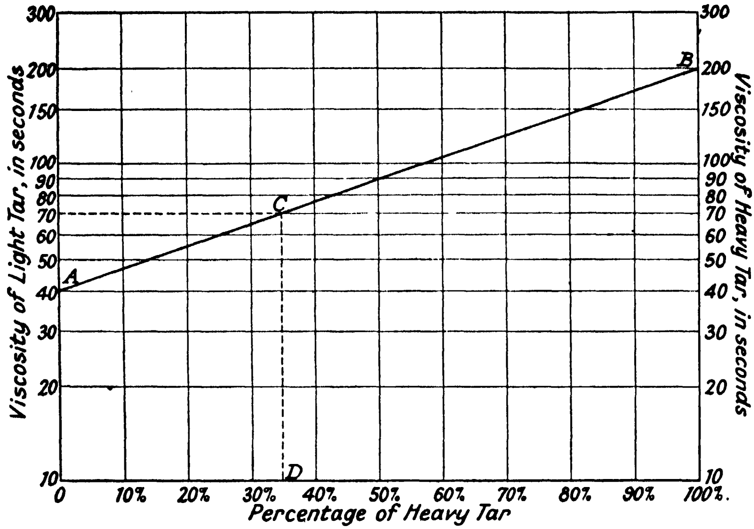

Fig. 15.—Viscosity of Tar Blends.

viscosity can be found by dropping a perpendicular from the point where this line cuts the required viscosity. The percentage is indicated at the point of the abscissa cut by this perpendicular. In the case in question, it has been assumed that the minimum and maximum viscosities are 40 and 200 seconds, and a line (*AB*) has been drawn connecting these viscosities. To find the proportions which will give a viscosity of, say, 70 seconds, the perpendicular *CD* is dropped, and the point *D* gives the percentage of heavy binder, in this case 35 per cent., which is needed to give a viscosity of 70 seconds, i.e., a mixture of 35 per cent. of a 200-viscosity tar plus 65 per cent. of a 40-viscosity tar will give a viscosity of 70 seconds.

As in the case of surface dressing, the quality of the tar used for the manufacture of tarmacadam is of the utmost importance. Remarks made in Chapter IV on the properties of the horizontal- and vertical-retort tars and the addition of bitumen apply equally to tarmacadam.

Bituminous Binders. There are a variety of bitumens to choose from as a binder for bituminous macadam, but they can be classified under four general groups:

1. Creosote fluxed bitumens.
2. Kerosene fluxed bitumens.
3. Bitumen fluxed with both kerosene and creosote oil.
4. Soft penetration bitumens.

The use of an unfluxed soft bitumen of penetration 200 to 500 involves the use of relatively high mixing temperatures of the order of a minimum of 170° F. (77° C.) and such materials must be laid warm. These will be dealt with under "Hot Process Materials".

Fluxed bitumens are supplied with varying viscosities so that the material can be laid cold, as in the case of tarmacadam, or, where a more rapid set is desired, a higher viscosity can be used, necessitating some degree of warmth in the material when laid.

The creosote fluxed bitumens would appear to be the most popular. These were usually supplied before the war in four grades, with viscosity limits approximately as follows:

Grade	B.R.T.A. viscosity at 30° C.
Light	70– 90 seconds
Medium	110–135 "
Heavy	155–190 "
Extra heavy	280–355 "

It should be noted that a testing temperature of 30° C. has been used in the above table for convenience of comparison with tar viscosities.

Many bitumen companies, although using the B.R.T.A. viscometer, have adopted a bitumen standard testing temperature of 25° C. for domestic reasons, and some confusion may arise if this point is overlooked.

At the time of writing only two grades are available:

Pool Cutback No. 1: 135–190 seconds at 30° C.
Pool Cutback No. 1A: 280–355 seconds at 30° C.

Manufacture.

(I) *Tarmacadam.* B.S. 802 was revised in 1945 and due consideration of improved technique has been made in the revision.

A new standard (B.S. 1242 : 1945) has now been issued. This deals with tarpaving separately, recognising the fact that material intended for surfacing of footpaths, play-grounds, etc., needs different characteristics from ordinary tarmacadam used for the surfacing of roads. The majority of tarpaving materials are made with limestone aggregate. Their use is so extensive that these materials are dealt with separately in Chapter VI.

With reference to B.S. 802 : 1945, the old distinction between cold, semi-hot, and hot process tarmacadam has been abandoned, and the subdivision of the different gradings is on the following basis :

(i) Materials to be used for base courses or single-course work (ordinary tarmacadam). See Table 18.
(ii) Materials to be used for wearing courses of open texture (ordinary tarmacadam). See Table 19.
(iii) Materials to be used for wearing courses of medium texture (preferably laid warm). See Table 20.
(iv) Materials to be used for wearing courses of close texture (to be laid hot). See Table 21.

The four tables giving the grading and binder tolerances for these materials are reproduced below. It should be clearly understood that the gradings specified are the gradings of the manufactured material and the figures do not represent bin percentages.

TABLE 18

Gradings and Binder Contents for Base Courses and Single Course

Sieve size (B.S. 410) Square aperture sieves	Nominal size		
	Base course		Single course
	2½ in.	1½ in.	1½ in.
	Percentage by weight passing		
2½ in.	100	—	—
2 "	90–100	100	100
1½ "	35–65	90–100	90–100
1 "	20–40	50–80	50–80
½ "	10–20	10–30	30–50
¼ "	—	—	20–30
No. 200	0–5	0–5	0–5
Quantity of binder—			
a. Percentage by weight of mixed material	3½ to 4½	3¾ to 4¾	4¼ to 5¼
b. Approximate equivalent gallons per ton of dry aggregate (including filler)	7 to 9	7½ to 9½	8½ to 10½

TABLE 19

Gradings and Binder Contents for Open-textured Wearing Courses (for 2- or 3-course Construction)

Sieve size (B.S. 410) Square aperture sieves	Nominal size		
	¾ in.	½ in.	⅜ in.
	Percentage by weight passing		
1 in.	100	—	—
¾ ,,	90–100	100	—
½ ,,	30–60	90–100	100
⅜ ,,	—	—	85–100
¼ ,,	10–20	25–45	30–60
⅛ ,,	0–10	5–15	10–20
No. 200	0–5	0–5	0–5
Quantity of binder—			
a. Percentage by weight of mixed material	4¾–5¾	5¼–6¼	5¾–6¾
b. Approximate equivalent gallons per ton of dry aggregate (including filler)	9½–11¾	10½–12¾	11¾–14

Table 20

Gradings and Binder Contents for Medium-textured Wearing Courses (preferably laid warm)

Sieve size (B.S. 410) Square aperture sieves	Nominal size			
	¾ in.	½ in.	⅜ in.	¼ in.*
	Percentage by weight passing			
1 in.	100	—	—	—
¾ "	90–100	100	—	—
½ "	—	90–100	100	—
⅜ "	45–65	—	90–100	100
¼ "	30–50	35–55	45–65	90–100
⅛ " †	15–30†	15–30†	15–30†	40–70
No. 200	4–6	4–6	4–6	0–10
Quantity of binder—				
a. Percentage by weight of mixed material	5¼ to 6¼	5½ to 6½	5¾ to 6¾	6¼ to 7¼
b. Approximate equivalent gallons per ton of dry aggregate (including filler)	10½ to 12¾	11 to 13½	11¾ to 14	12¾ to 15

* This grade has been included in this table for convenience and is intended for use primarily for patching, etc.

† The content passing ⅛-in. sieve should be kept as high as practicable, and of this material at least 40 per cent. shall pass a No. 14 sieve.

TABLE 21

Gradings and Binder Contents for Close-textured Wearing Courses (to be Laid Hot)

Sieve size (B.S. 410) Square aperture sieves	Nominal size 1 in.	Nominal size ½ in.
	Percentage by weight passing	
1½ in.	100	—
1 ,,	90–100	—
¾ ,,	60–80	100
½ ,,	45–65	90–100
¼ ,,	30–50	40–50
⅛ ,, *	25–35*	25–35*
No. 200	5–10	5–10
Quantity of binder—		
a. Percentage by weight of mixed aggregate	5¾ to 6¾	6¼ to 7¼
b. Approximate equivalent gallons per ton of dry aggregate (including filler)	11¾ to 14	12¾ to 15

* Of this material at least 40 per cent. shall pass a No. 14 sieve.

The types and viscosities of tar binder specified for the above gradings are shown in Table 22.

TABLE 22

Types and Viscosities of Tar Binder

Construction	Type of tar	Viscosity of tar expressed as E.V.T. and as seconds at particular temperatures			
		Summer		Winter	
		E.V.T. °C.	Viscosity sec.	E.V.T. °C.	Viscosity sec.
Base course and single course as in Table 18	A or B	34–38	45–80 at 35° C.	27–34*	30–100 at 30° C.
Open-textured wearing courses as in Table 19	A or B	34–38	45–80 at 35° C.	27–34*	30–100 at 30° C.
Medium-textured wearing courses as in Table 20 (preferably laid warm)	B or C	37–41	70–140 at 35° C	34–37	45–70 at 35°C.
Close-textured wearing courses as in Table 21 (to be laid hot)	A or B	50–55	50–120 at 50° C	—	—

* For base course, single-course, and open-textured wearing course material dispatched by rail in cold winter weather, it may be advisable to reduce the E.V.T. from 27° to 24° C. This will apply only to type A tar.

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When considering Table 22 it should be borne in mind that the viscosity limits are not all based on the normal testing temperature of 30° C. For example, the summer viscosity for medium-textured wearing course is given as 70–140 seconds at 35° C. This is very approximately equivalent to 170–400 seconds at 30° C., in other words, a relatively high-viscosity tar. The tar specified for the close-textured wearing course is tested at 50° C., and the corresponding test at 30° C., if it were possible to carry it out, would give a reading of the order of 2,500–8,000 seconds at 30° C.

The use of tar bitumen mixtures is also foreseen and the binder may, by agreement between the tarmacadam manufacturer and the purchaser, consist of one of the following mixtures:

(i) Tar to B.S. 76, types A and B, and asphaltic bitumen, defined as follows:

Asphaltic Bitumen. Natural or naturally occurring bitumen, or bitumen prepared from natural hydrocarbons or from derivatives of natural hydrocarbons by distillation or oxidation or cracking; solid or viscous, containing a low percentage of volatile products; possessing characteristic agglomerating properties, and substantially soluble in carbon disulphide.

(ii) Tar to B.S. 76, types A and B, and natural asphalt, defined as follows:

Natural Asphalt. Mixtures occurring in nature, in which asphaltic bitumen is associated with inert mineral matter.

(iii) Tar to B.S. 76, types A and B, and lake asphalt, defined as follows:

Lake Asphalt. An asphalt which, as found in nature, is in a condition of flow or fluidity.

The proportions of the ingredients and the characteristics of the mixture shall be agreed between the tarmacadam manufacturer and the purchaser.

The temperatures of aggregate and tar binder specified are given in Table 23.

TABLE 23

Temperatures of Mixing

Construction	Temperature of aggregate		Temperature of binder	
	Min.	Max.	Min.	Max.
Base course, single course, and open-textured wearing course with tar having an E.V.T. 24–34° C.	—	110° F. (44° C.)	140° F. (60° C.)	200° F. (93° C.)
Base course, single course, and open-textured wearing course with tar having an E.V.T. 34–38° C.	100° F. (38° C.)	140° F. (60° C.)	140° F. (60° C.)	220° F. (104° C.)
Medium-textured wearing courses with tar having an E.V.T. 34–41° C.	120° F. (49° C.)	160° F. (71° C.)	140° F. (60° C.)	220° F. (104° C.)
Close-textured wearing courses with tar having an E.V.T. 50–55° C.	190° F. (88° C.)	230° F. (110° C.)	180° F (82° C.)	240° F. (115 C.)

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It will be noted that no minimum aggregate temperature is given for base courses or wearing courses which are to be laid cold. The proviso is however made under paragraph 10, that the aggregate shall be thoroughly dry, which is an important consideration, as even the high adhesion value of limestone will not ensure satisfactory results if damp or wet aggregate is coated.

The gradings shown under Table 20 for medium-textured wearing courses are worthy of special comment, as they are very much in accordance with the recommendations of the Road Research Laboratory [1] following a large number of small-scale trials. The best of these trial sections have stood up to more than five years' heavy main-road traffic without requiring attention. This would appear to be due largely to the carefully designed grading, which has a very low void content. This type of material is preferably laid warm.

The gradings shown under Table 21 deal with close-textured materials with a low void content which must be laid hot and are rather outside the range of ordinary tarmacadam. Manufacture of this class of material is usually very difficult, if not impossible, with an ordinary tarmacadam plant.

(II) *Bituminous Macadam.* There is no British Standard dealing with bituminous macadam in its present-day accepted form. B.S. 510 and B.S. 511, which at the moment only cover a few proprietary bituminous carpetting materials, are however in the process of revision and extension to include bituminous macadam and cold laid bituminous carpets.

A fluxed bitumen is usually employed as the binder for bituminous macadam although in some cases a penetration bitumen is used, the necessary fluxing oil being added at the mixer. There are many reputable brands of proprietary fluxed bitumens on the market. Excellent results have been obtained with the creosote or mixed fluxed types, especially if any difficulty is experienced in obtaining thoroughly dry aggregate.

When coating stone with a fluxed bitumen, it should be borne in mind that the bitumen, after the evaporation of the fluxing oils, does not harden by oxidation and other chemical changes to the same degree as is the case with a tar binder.

Bitumens, owing to their prolonged life in a plastic state, can rise more readily to the top in hot weather, and a slippery film of bitumen can easily form on the surface if too much bitumen has been used in the mix. For this reason, the amount of bitumen used should be as low as is consistent with durability.

[1] War-time Road Note No. 7. (H.M.S.O.)

The gradings used for the manufacture of tarmacadam or tar carpets can be used with equal success with bituminous binders. The gallonages shown for the tar products will, however, be rather on the high side and should be reduced by about 10 per cent. when using fluxed bitumen. Aggregate temperatures are approximately the same as for tarmacadam, but the binder temperatures, particularly in the case of the heavy and extra heavy grades, have to be increased up to 260° F. (126° C.).

Laying of Tar and Bituminous Macadam.

Choice of Specification. The use of the correct size and grading for the job is important, and B.S. 802 : 1945 makes recommendations shown in Table 24 :

Foundations. These must be strong enough to carry the anticipated traffic. Trouble will be experienced if the foundations are weak, no matter how good the surfacing material.

Lateral Support. This is an essential feature, which is often omitted on account of cost. Absence of lateral support soon leads to fraying and fretting of the edges of the road. This can extend to potholes and corrugations, and general unevenness may develop. The effective road width is thus seriously reduced, as cyclists will be forced to ride well away from the edges.

Key. Some means of preventing the surfacing from slipping on the foundations is necessary, particularly if these are very smooth. Pushing under traffic, with subsequent waving or corrugations, will otherwise result. Most foundations are rough enough, or have sufficient "bite" to prevent any such movement, but if a "key" of emulsion is required as a safeguard, this should be applied with care. Any excess can easily creep up through the surfacing, particularly if the surfacing to be applied is thin. Normal spraying emulsions are really too thick and viscous to be spread thinly enough. Specially thinned "Tack-coat" emulsions have been recently developed for this purpose, and their use is recommended. Spreading capacities of 12–25 sq. yds. per gallon can be obtained. The surface should be well swept before applying the emulsion, and laying should be deferred until the emulsion has broken, i.e., turned black.

Tipping Material. The practice of trying to get the lorry to do most of the spreading by tipping the load in a series of small heaps is unsound, and should on no account be tolerated. Loads should

TABLE 24

Gradings for Various Consolidated Thicknesses of Tarmacadam

Number of courses	Range of consolidated thicknesses for which the type of construction is suitable	Thickness of consolidated course (or courses)	Nominal size of tarmacadam
One	2 in. to 3 in.	2 in. to 3 in.	1½-in. single course
Two	3 in. to 4 in.	Base course 2¼ in. to 3¼ in. Wearing course ¾ in. to 1 in. 1 in. to 1½ in.	1½-in. or 2¼-in. base course ⅜-in. or ½-in. open-, medium-, or close-textured course ½-in. or ¾-in. open-, medium-, or close-textured course
Three	4 in. min.	Base course 2½ in. min. Intermediate course 1½ in. minimum Wearing course ¾ in. to 1½ in.	2¼-in. base course 1½-in. base course ⅜-in., ½-in., or ¾-in. open-, medium-, or close-textured course
	Wearing courses (medium- and close-textured) for single-course work		
One	¾ in. to 2 in.	¾ in. to 1 in. 1 in. to 1½ in. 1½ in. to 2 in.	⅜-in. or ½-in. medium-textured course ½-in. or ¾-in. medium-textured course, or ½-in. close-textured course 1-in. close-textured course

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be tipped in one heap well back from the area to be surfaced, and the material carted in barrows to the job as required. The material should be protected by a sheet during inclement weather. This method of laying has the following advantages:

(*a*) Segregation during unloading is greatly reduced.
(*b*) Any segregation which might have occurred can be rectified when loading into barrows.

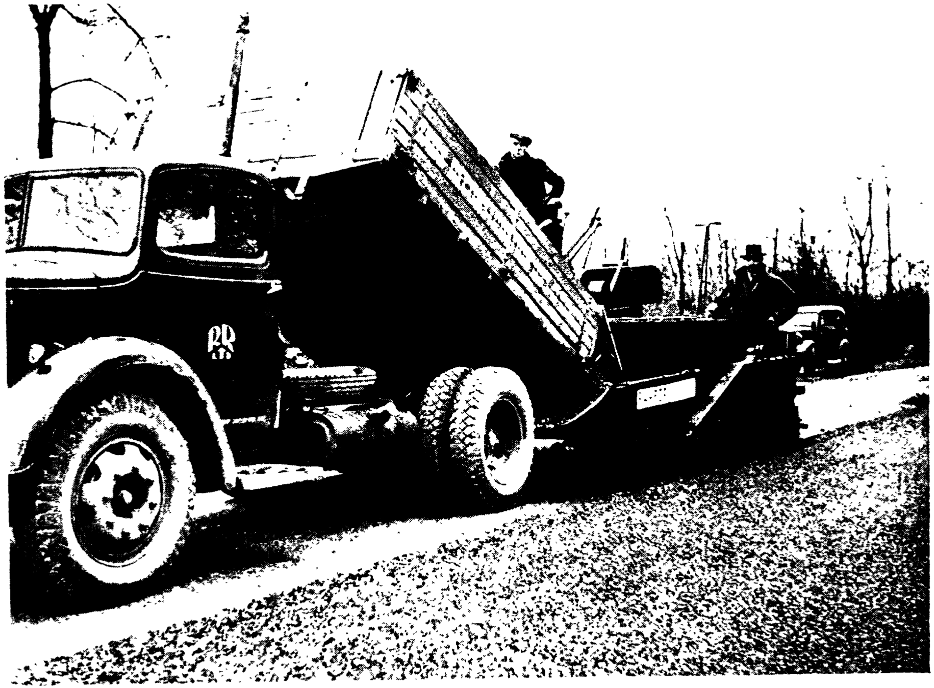

Fig. 16.—Road-surfacing Machine spreading $1\frac{1}{2}$-in. Graded Tarmacadam.

[*Facing p. 76.*

Fig. 17.—1½-in. Graded Bituminous Limestone laid on A.38 (Woodhouse Down) in 1932–33. Photographed December 1944.

Reproduced by permission of E. C. Boyce, Esq., B.Sc., A.M.Inst.C.E., County Surveyor, Gloucestershire County Council.

[Facing p. 77.

(*c*) Material does not cool off so quickly, and can be readily protected from a sudden rainstorm.
(*d*) Men work on a clean surface instead of trampling on surplus material, which can have the effect of producing an uneven surface.

Laying. If dirty, the surface should be swept with a stiff broom. In the case of single-coat work, big depressions or potholes should be filled in a few days before actual laying commences. The material should be spread uniformly and evenly compacted with forks. The final finish of toppings can be best obtained with a rake. Great care should be taken to prevent the intrusion of foreign matter such as wood (including matches), rubber, and animal droppings. The last are exceedingly dangerous, and should be thoroughly removed from the job by scraping and sweeping. If any manure gets into the base coat it will be found best in the long run to cut out the contaminated area and replace with fresh material.

A war-time innovation which will probably persist after the war is the use of mechanical spreading and finishing machines. These are, at the moment, all of American design, and Fig. 16 illustrates a popular type. Advantages claimed which have been proved in practice are:

(*a*) Big capacity (up to 500 tons per day).
(*b*) Reduction in spreading and rolling costs.
(*c*) Provision of a very even surface.

Camber. A cross-fall not exceeding 1 in 40 is specified in B.S. 802 : 1945. This should provide adequate drainage for the surfacings described in this chapter. Fine-graded carpets or asphalt surfacings can be laid with as little cross-fall as 1 in 48. The cross-fall necessary to ensure adequate drainage will depend on the longitudinal fall and should always be as low as possible.

Spreading Capacity. Table 25 is reproduced from B.S. 802 : 1945, and it should be noted that the rates given are only approximate. B.S. 802 : 1945 points out that the table is given without implying obligation on any of the parties concerned and that the tolerances given take into account the fact that the covering capacity per ton is influenced by such factors as specific gravity of aggregate, grading of aggregate, weather conditions, length of haul, nature and/or condition of foundations, whether the material is laid in one or more courses, weight of roller, etc.

TABLE 25

Rates of Coverage for Tarmacadam

Thickness of course	Grading (see Tables 18, 19, 20 and 21)	Approximate covering capacity
in.		sq. yd. per ton
½	⅜-in. open-textured wearing course or ⅜-in. medium-textured wearing course	38–42 38–42
¾	½-in. or ⅜-in. open-textured wearing course, or ½-in. or ⅜-in. medium-textured wearing course	25–30 24–28
1	¾-in., ½-in., or ⅜-in. open-textured wearing courses, or ¾-in., ½-in., or ⅜-in. medium-textured wearing courses, or ½-in. close-textured wearing course	22–25 21–24 19–22
1½	¾-in. open-textured wearing courses, or ¾-in. medium-textured wearing courses, or 1-in. or ½-in. close-textured wearing courses	15–18 14–17 14–17
2	1½-in. base course 1½-in. single course 1-in. close-textured wearing course	12–14 12–14 11–13
2½	2¼-in. or 1½-in. base course 1½-in. single course	10–12 10–12
3	2¼-in. or 1½-in. base course 1½-in. single course	8–10 8–10
3¼	2¼-in. or 1½-in. base course	7–9

In the case of limestone, very little fluctuation of specific gravity occurs, and an average figure of 2·69 can be taken.

A few outstanding cases of successful tar and bituminous surfacing work have been mentioned in Chapter III. A further case worthy of special mention is the section of road illustrated in Fig. 17. This road has stood up to ten years of very heavy main-road traffic (traffic at last census was over 10,000 tons per day), and it is understood that no surface dressing or maintenance work of any description has been necessary. The material used was an orthodox 1½-in. graded hard Carboniferous mountain limestone, coated with a creosote fluxed bitumen.

There is no doubt that limestone occupies a pre-eminent position as an aggregate for these types of surfacings, and the goodwill which has been established is undoubtedly due to the unique physical and chemical characteristics of limestone, which give that extra latitude in manufacture so necessary for a process where precise and complete standardisation is not possible on economic grounds.

CHAPTER VI

TAR AND BITUMINOUS PAVING MATERIALS

Special Considerations—Official Tests—Tarpaving—Bituminous Paving—Laying and Rolling—Spreading Capacity

UNDER the description of paving materials, it is necessary to include not only materials used for the surfacing of footpaths, but similar materials used for similar forms of construction, such as surfacing of playgrounds, tennis courts, drives, etc.

SPECIAL CONSIDERATIONS

A proper appreciation of the difficulties of this form of construction explains the necessity of using special materials.

The chief difficulties which arise in connection with paving materials are:

1. Foundations. The mistake is often made of neglecting the foundations on the false assumption that a very light foundation is sufficient to carry foot traffic. Although the same strength of foundations as for a road is unnecessary, it must be appreciated that the paving material will have to be consolidated by rolling. The foundation must be strong enough not only to carry the weight o the roller, but also to offer sufficient resistance to enable complete compaction to take place. The foundation must also be strong enough to carry occasional abnormal weights, such as a loaded wheelbarrow.

2. Rubbling or Fretting. Foot traffic has not the ironing and facing-up effect exerted by road traffic, where the weight of wheeled traffic ensures relatively rapid consolidation and closing up of the voids. A much longer period is necessary in the case of foot traffic to attain the same result, and a footpath surfacing material, unless specially designed for the job, would remain open and be thus subject to the adverse effects of weather for a longer period.

Most of the successful paving materials in use undoubtedly owe their success in the first place to the use of limestone as an aggregate, coupled with the use of a grading containing a minimum of voids consistent with reasonable ease of spreading.

Special difficulties arise in connection with paving materials used for the surfacing of playgrounds as the traffic stresses are very severe.

For example, the sliding of hobnailed boots on the surface will readily find any weak spots. In addition, it is necessary to avoid the use of an aggregate which could in the event of cuts or grazes induce sepsis. Limestones are quite safe in this respect.

A relatively high binder content is necessary to see this type of material through the early stages until it is completely consolidated by traffic, and it is also necessary for the binder to be firmly held by the stone so that no stripping takes place in adverse weather conditions shortly after laying. The successful work carried out using limestone paving materials is no doubt largely due to the retentive surface texture and high adhesion value of limestone.

The use of a hard limestone is not so necessary for this class of work, and the softer limestones which would normally be unsuitable for the surfacing of roads carrying heavy traffic can be used for footpath work.

OFFICIAL TESTS

Limestone paving materials occupy their predominating position largely due to the excellent results obtained. These have been confirmed by experiments carried out by the Ministry of War Transport. The ¾-in. limestone section forming part of the Ministry test carried out at Swainnow Lane using various types of paving construction is reported [1] to have given the best results of the seven materials laid there, and reference is made to good results of tarred limestone used in another experiment on footpaths of the Kingston By-pass.

TARPAVING

The publication of B.S. 1242 recognises the importance of tar-paving materials by standardising them for the first time and dealing with them separately from tarmacadam. The chief differences to be noted between tarpaving and ordinary tarmacadam materials are as follows:

(1) Higher gallonage.
(2) Closer grading for the wearing surface.
(3) Lower viscosity tars.
(4) Slightly lower mixing temperature.

The necessity for the higher gallonage and closer grading has already been explained. Lower viscosity tars than for road con-

[1] *Experimental Work on Roads*, Report for 1937–38, p. 162. (H.M.S.O.)

struction have to be used, chiefly because consolidation has to be effected with lighter rollers than are usually employed for roads. The lower mixing temperatures are specified in order to avoid the possibility of the material setting off prematurely with consequent spreading and rolling difficulties.

Two sizes only are standardised, a 1-in. graded material for the base course, and a $\frac{3}{8}$-in. graded down to dust for the wearing course. The grading and binder requirements are reproduced in Table 26.

TABLE 26

Gradings and Binder Contents for Base Course and Wearing Course

Sieve size (B.S. 410) Square aperture sieves	Nominal size: Base course 1 in. — Percentage by weight passing	Nominal size: Wearing course $\frac{3}{8}$ in. — Percentage by weight passing
$1\frac{1}{4}$ in.	100	—
1 ,,	90–100	—
$\frac{3}{4}$,,	45–65	—
$\frac{1}{2}$,,	—	100
$\frac{3}{8}$,,	5–15	90–100
$\frac{1}{4}$,,	—	60–80
$\frac{1}{8}$,,	—	30–50*
No. 200	—	0–5
Quantity of binder—		
a. Percentage by weight of mixed aggregate	4–5	$6\frac{1}{4}$–$7\frac{1}{4}$
b. Approximate equivalent gallons per ton of dry aggregate (including filler) .	8–10	13–15

* Of the material passing $\frac{1}{8}$-in. sieve, at least 30 per cent. shall pass a No. 14 sieve.

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Type A or B tars are specified and the viscosity requirements are shown in Table 27.

The mixing temperatures are given as a maximum of 100° F. (36° C.) for the aggregate and 200° F. (93° C.) for the binder.

TABLE 27

Types and Viscosities of Binder

Course	Type of tar	Viscosity, expressed as E.V.T. in ° C. and as seconds at 30° C.			
		Summer		Winter	
		E.V.T.	Viscosity	E.V.T.	Viscosity
Base course . .	A or B	30°–34°*	50–100*	24°–30°†	20–50†
Wearing course .	A or B	27°–31°*	30–60*	20°–27°	10–30

* In very hot weather it may be advisable to increase the viscosity for the base course up to 36° C. E.V.T. (150 sec. at 30° C.) and for the wearing course up to 34° C. E.V.T. (100 sec. at 30° C.).

† When limestone is used the lower limit may be reduced to 20° C. E.V.T. (10 sec. at 30° C.).

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BITUMINOUS PAVING MATERIALS

Equally good results are obtainable, using fluxed bitumens a a binder and the gradings specified in B.S. 802 can be followed. A reduction of up to 10 per cent. in the binder content, however, is desirable. The lighter grades of fluxed bitumens should be used for this class of material.

LAYING AND ROLLING

Foundations. B.S. 1242 does not specify the type of foundation to be used, except to say that the foundation shall be suitably prepared.

It is difficult to standardise the foundations for such surfacings, as conditions vary greatly and there is usually a very wide choice of materials which can be used. The following specification is suggested as being fairly representative:

(i) If the subsoil consists of earth, at least 4 in. of broken stone, gravel, clinker, or hardcore should be used. If clay is underneath and this cannot be removed on account of cost, an insulating layer of 4 in. of fine clinker should be used. The foundations should be laid true to shape and well rolled with a roller of at least 2-ton weight. In any event, the weight of the roller must be at least as great as that of the roller used for the consolidation of the tarpaving material. Any depressions made by the roller or by settlement should be made up with foundation material before laying the coated stone.

(ii) *Lateral Support.* This is just as important as in the case of road construction, and should be provided wherever possible.

(iii) *Spreading and Consolidating.* Material should be spread uniformly and evenly compacted with forks. The final finish of the wearing course can be best obtained with a rake. Great care should be taken to prevent the intrusion of foreign material such as wood (including matches), rubber, and manure. B.S. 1242 specifies that the surface shall conform to the level and cross-section specified within a limit of $\frac{3}{8}$ in. in 10 ft., or such other limits as may be agreed with the engineer. A maximum weight for the roller of 50 cwt. is foreseen. This would appear to be an unnecessary limitation in the event of the foundations being strong enough to take a heavier weight of roller.

(iv) *Dusting of Surface.* B.S. 1242 recommends that the surface of the tarpaving shall be lightly dusted with $\frac{1}{8}$-in. graded down coated material, or with a fine dry dust not exceeding $\frac{1}{8}$ in. graded down, and it is recommended that the surface should be lightly rolled after dusting.

SPREADING CAPACITY

B.S. 1242 says that the tarpaving shall be laid in one course or more as specified by the engineer. The grading for various consolidated thicknesses are reproduced in Table 28. It is further specified that the base course shall have a thickness not less than $1\frac{1}{4}$ in. or greater than 2 in. The wearing course, where applied on a new base course, shall be of a thickness not less than $\frac{1}{2}$ in. or greater than $\frac{3}{4}$ in. In cases where the wearing course is laid as a single course on an existing foundation, the thickness shall be not less than $\frac{3}{4}$ in. or greater than $1\frac{1}{2}$ in.

Table 28A, reproduced from B.S. 1242, gives a very approximate indication of the spreading capacity of paving materials laid to varying thicknesses. It should be noted that this table is given without implying obligation on any of the parties concerned. It will also be seen that the spreading capacities for paving materials are somewhat higher than those shown in the British Standard for road materials. Part of the difference between the two sets of figures can be explained by the fact that consolidation of paving materials cannot usually be as thorough as is the case with road-surfacing materials.

TABLE 28

Gradings for Various Consolidated Thicknesses of Tarpaving

No. of courses	Range of consolidated thickness for which this type of construction is suitable	Thickness of consolidated course or courses	Nominal size of tarpaving
1	¾ in. to 1½ in.	¾ in. to 1½ in.	⅜-in. wearing course
2	1¾ in. to 2¾ in.	Base course 1¼ in. to 2 in. Wearing course ½ in. to ¾ in.	1-in. base course ⅜-in. wearing course

TABLE 28A

Rates of Coverage for Tarpaving

Thickness of course	Grading (see Table 26)	Approximate covering capacity
in.		sq. yd. per ton
½	⅜-in. wearing course	38–42
¾	⅜-in. ,, ,,	26–30
1	⅜-in. ,, ,,	22–26
1¼	1-in. base course	24–28
1½	1-in. ,, ,,	17–20
2	1-in. ,, ,,	14–16

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CHAPTER VII

LIMESTONE CARPETS

Definition—Importance of Adhesion—Stone Mixtures—Manufacturing Plant —Coarse- and Medium-Textured Carpets—Fine-Textured Carpets—Laying of Carpeting Materials—General

Definition. A carpet can be defined as a mixture of aggregate and binder laid on a road surface to a thickness not appreciably exceeding 1 in., whose purpose is to restore the good riding qualities of a road which is sound underneath, but whose surface has deteriorated and needs attention. This will be due to one or more of the following reasons:

(1) The existing surface is uneven, corrugated or wavy. These conditions often exist if a road has been surface dressed too many times or if the surface dressings have been too " fat ".

(2) The top surface of the road is beginning to show signs of wear. In this case it is important to differentiate between normal wear and damage caused by weak foundations. If the latter is the case, caution must be exercised before considering the application of a carpet. Carpets, owing to their thinness, have no bridging strength and cannot be expected to strengthen the foundations. These, if weak, must either be strengthened locally or a heavier form of construction should be chosen.

(3) Carpets are often used to finish off the widening and reconstruction of a narrow and badly shaped road. In this case a combination of two-coat and carpeting work is usually employed, the base coat being used for those areas which require most making up.

(4) Carpets are often used to give a non-skid surface to roads which have become slippery.

Carpeting materials are also used as the wearing surface of a newly laid base coat.

Carpeting materials are often used in cases where surface dressing would previously have been employed, and there is a growing tendency for carpets to supplant surface dressing, especially on main roads.

Cost. It is difficult to give an accurate average cost of laying a carpet and to compare this with the cost of a surface dressing as an alternative. It is, however, safe to assume that surface dressing properly carried out during suitable weather conditions with an assumed life of three years, would be cheaper on a cost per square yard per year basis if a five-year life were taken as the nominal life of a carpet.

It must, however, be borne in mind that with our climate, weather conditions suitable for surface dressing do not often prevail, and if the average results obtained by surface dressing are compared with carpets, there is no doubt that the latter are more economical on a cost per square yard per annum basis.

There is also a limit to the number of surface dressings which can be applied to a road surface, and it is impossible to remove depressions and restore the riding qualities of an uneven road by merely surface dressing. In addition, a surface dressing shortly after application and particularly in hot weather can be a source of great inconvenience to traffic.

Types of Carpet. Carpets can be divided broadly into three classes, as follows:

(*a*) *Coarse Texture.* $\frac{3}{4}$-in. or $\frac{1}{2}$-in. single-sized stone, or graded down to $\frac{1}{4}$ in., but still containing an appreciable amount of voids.

(*b*) *Medium Texture.* $\frac{3}{4}$-in. or $\frac{1}{2}$-in. stone, well filled with smaller sizes, and containing sufficient filler or quarry fines to give an appreciably voidless mix. B.S. 802 defines a further type of carpet under the heading " Close Texture ", but as the line of demarcation between materials of this type and the medium-textured carpets is not too clear, it is proposed to include both under the heading " Medium Texture ".

(*c*) *Fine Texture.* $\frac{1}{4}$-in. or $\frac{1}{8}$-in. down quarry fines.

These classifications can be subdivided further according to the viscosity (or penetration) of the binder used, resulting in materials which can be laid cold or have to be laid warm or hot.

Asphaltic bitumen is chiefly used as a binder, but intensive research work has been carried out by the Tar Industry, and there are now a number of standard tars on the market which are claimed to give satisfactory results as a carpet binder.

Importance of Adhesion. All types of aggregates have been tried with varying results, and there is no doubt that the choice of an aggregate possessing maximum adhesion towards the binder is of great importance in the case of carpeting materials.

Carpets are often laid on relatively impervious bases, sometimes with very little key, and in the case of a cold-lay carpeting material, a relatively high percentage of voids will be present, which will allow rain to get through the surface without being able to penetrate further. The protection given by surface dressing or a sealing coat is usually denied a carpet, as the elimination of surface dressing is one of the chief reasons for using carpeting materials. The action of traffic on a waterlogged carpet after rain can be very severe, and failure of the carpet will result unless the binder adheres adequately to the stone. Because of its good adhesion limestone is an excellent aggregate for all classes of carpets.

Stone Mixtures. Tests have shown that the stability of a carpet made with a low adhesion value aggregate can be improved by incorporating limestone chippings and/or dust in the grading. The minimum amount of limestone necessary to achieve a definite improvement appears to be at least one-third of the whole.

There is a tendency based on considerations of first cost to use local aggregate as far as possible, but if this aggregate should have a low adhesion value, it may be possible to use it successfully by incorporating limestone in the grading.

Manufacturing Plant. Owing to the severe stresses imposed upon carpets, and the necessity for maximum efficiency, it is essential to manufacture such materials at coating units which permit accurate proportioning of the ingredients and close control of the temperatures. Special plants for the manufacture of carpets have been designed, but there is no reason why a well-equipped tarmacadam plant should not be able to manufacture this class of material.

The chief requirements are:

(*a*) Means of gauging accurately the different stone sizes used in the grading.

(*b*) Means of measuring accurately the batch of stone and amount of binder used. Measurement by weight, as explained in Chapter V, is better than measurement by volume.

(*c*) An efficient drier which will completely dry the aggregate without exceeding the maximum stone temperature, or alternatively, the provision of adequate cooling-off bins.

(*d*) An efficient mixer, preferably of the paddle type. In the case of fine-textured carpets the discharge door should be a tight fit, material should not be able to leak out of the mixer, and the paddle tips should clean out the material completely after discharging the batch (cf. Fig. 18).

(*e*) Accurate control of binder temperature.

Coarse- and Medium-Textured Carpets. B.S. 802 deals with tar carpets and B.S. 510 with carpets made with asphaltic bitumen.

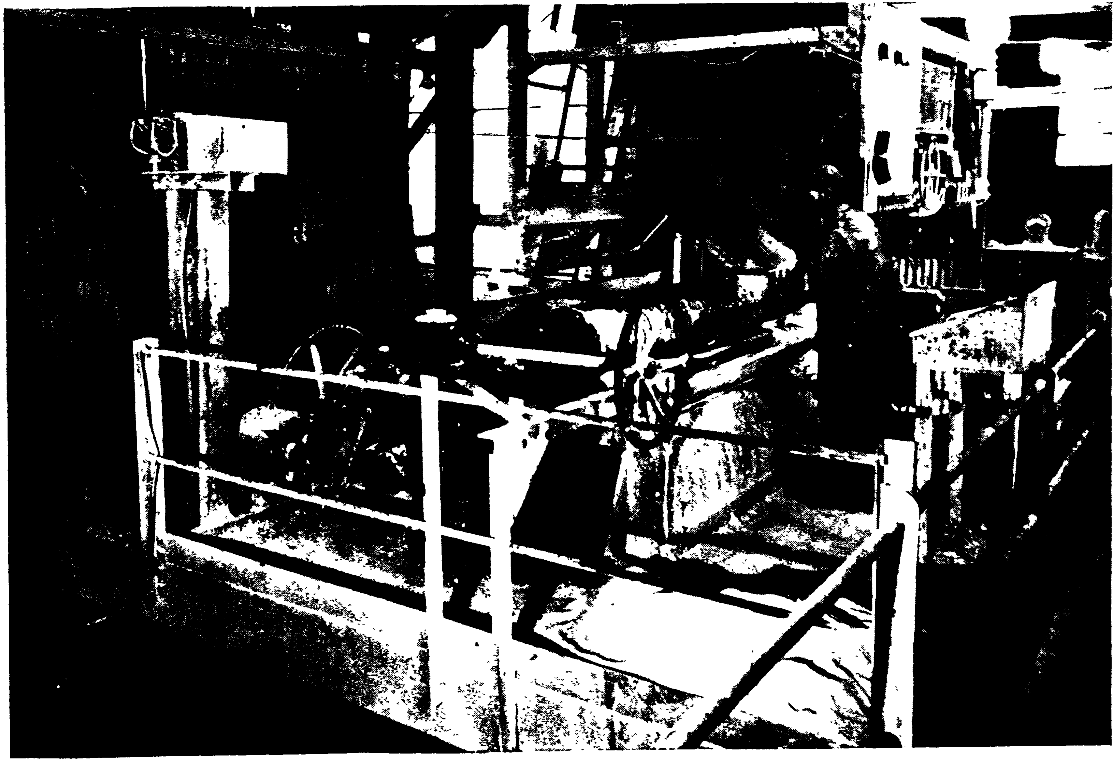

Fig. 18.—A High-capacity Coated Macadam and Carpet Travelling Mixer.

[Facing p. 88.

B.S. 802 gives a range of three sizes for medium-textured tar carpeting materials, the nominal sizes being $\frac{3}{4}$ in., $\frac{1}{2}$ in., or $\frac{3}{8}$ in. Two nominal sizes are specified for close-textured carpets, namely, 1 in. and $\frac{1}{2}$ in. It should be noted that this type of material must be laid hot. Full details of grading, binder content, and temperature requirements are given in Chapter V.

B.S. 510 (dealing with single-coat cold asphalt) is the only British Standard dealing with carpeting materials made with asphaltic bitumen. This specification is on very general lines and, in many instances, not in accordance with standard practice, and this class of carpeting material is rarely specified in accordance with B.S. 510. This specification is being revised, and the revision will doubtless be in accordance with modern practice.

The majority of proprietary fluxed asphaltic bitumens on the market are very similar in their characteristics. These can be conveniently grouped under two main heads:

(1) Penetration bitumen (usually 200–500 pen.) for warm or hot-lay carpets. The temperature of the manufactured material is of the order of 80°–100° C. (176°–212° F.)

(2) Cutback asphaltic bitumens for warm or cold-lay materials. These can be further subdivided into kerosene cutback type and creosote cutback type. A few manufacturers use a mixture of kerosene and creosote.

Kerosene cutback can be used with limestone, but with stones having poor adhesion better results would be obtained with a creosote cutback. The exact reason for the better adhesion obtained with a creosote or tar-oil fluxed bitumen is not yet clearly established, although there is little doubt that the tar acids play an important rôle. It should however be noted that satisfactory results cannot be expected even with a creosote cutback bitumen when using a stone with a Riedel and Weber adhesion value of 0 or 1.

The viscosity requirements of pre-war bitumens and the pool war grades outlined in Chapter V apply equally to carpeting materials.

The coarse-textured type of carpet which can consist of single-sized stone coated with 200 to 500 pen. bitumen was not extensively employed before the war. The good results obtained during the war with so-called " pervious asphalt " may set up a new post-war fashion. One drawback is that this class of material necessitates relatively high mixing and laying temperatures.

A great number of proprietary carpeting materials existed before the war, which come under the heading of " medium texture ".

These usually consisted of a $\frac{3}{4}$-in. or $\frac{1}{2}$-in. graded-down aggregate coated with a fluxed bitumen at the rate of approximately 10 gallons per ton, and very successful results have been obtained using limestone, or a mixture of limestone and igneous rock, as the aggregate.

Another process is to coat the aggregate with a special flux oil, to which is then added a hard powdered bitumen and filler.

The material obtained remains quite lively until spread and rolled when consolidation brings the film of oil and the particles of hard bitumen into close contact so that amalgamation of the two gradually takes place under traffic. The resultant surface presents a closely knit mosaic comparable in appearance to that of a hot asphalt. This manufacturing process can be used also for fine texture carpets.

A proprietary carpeting material of this type has been used with considerable success and has stood up to fifteen years of heavy traffic without any further treatment, and is shown in Figure 19.

Fine-Textured Carpets. There are many varieties of fine-textured carpets on the market, and these can be roughly classified as follows:

(*a*) A low-viscosity fluxed bitumen is used, the fluxing oil being essentially volatile. Stability of the carpet is obtained by evaporation of these oils. Special tar binders have also been used successfully.

(*b*) The aggregate is mixed with a relatively high viscosity fluxed bitumen, the fluxing oil employed being essentially non-volatile. A relatively low percentage of bitumen is used, so that each particle is coated with a very thin film of binder, the principal rôle of which is to act as a lubricant. This enables the stone particles to interlock readily, so that a nearly voidless surface is obtained.

(*c*) The aggregate is coated with a high-viscosity fluxed bitumen, rapidly cooled as it is discharged from the mixer so that the particles do not stick together, and the resulting product is mixed with a very light and volatile fluxing oil on the site in a concrete mixer.

As most of the above classes of fine-textured carpets are covered by patents, no attempt will be made to reproduce any working specifications.

In addition, the chief constituent is quarry fines ($\frac{1}{8}$ in. or $\frac{1}{8}$ in. plus $\frac{1}{16}$ in. down), and the grading of these varies considerably from quarry to quarry. It is accordingly impossible to give details of any one specification which could cover the wide range of aggregates likely to be used. Any such information would be misleading and probably do more harm than good.

Fig. 19.—Limestone Cold Asphalt Surfacing on A.27, Chichester–Arundel Road, near Crocker Hill. Laid June 1930. Photographed 1945.

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[*Facing p.* 90.

Fig. 20.—$\frac{3}{4}$-in. Graded Bituminous Carpet (Mountain Limestone Aggregate) laid on A.38 in 1938. Photographed December 1944.

Reproduced by permission of E. C. Boyce, Esq., B.Sc., A.M.Inst.C.E., County Surveyor, Gloucestershire County Council.

[*Facing p.* 91.

Laying of Carpeting Materials. The instructions given in Chapter V for tar and bituminous macadam apply equally well to carpeting materials. It is accordingly not proposed to detail these again, but to stress the following points:

Foundations. These must be strong enough to carry the anticipated traffic.

Key. Carpets are often applied to relatively smooth surfaces which call for some type of mechanical bond with the carpeting material. A cold emulsion applied as thinly as possible will give good results, but it is very important to avoid the use of any excess which could creep through to the top. The specially diluted " Tack Coat " emulsions should preferably be used. The writer has found the following alternative specification reliable and safe:

Clean the existing road surface, apply a normal emulsion fairly thinly (about 8 to 10 sq. yds. per gallon). As soon as the emulsion has broken (i.e., turned black), grit with $\frac{1}{8}$ in. clean limestone chippings. Roll with a heavy roller. Brush lightly the following morning to remove any excess limestone grit. A thin coat of bitumen and limestone, which affords an excellent key, will be left on the road. The use of the limestone chippings will prevent any extrusion of bitumen emulsion.

This specification has also been successfully used to provide a key for the carpeting of a road surfaced with granite setts. The sett joints should be cleaned of all loose material to a depth not exceeding $\frac{3}{4}$ in., after which a somewhat more liberal dressing of emulsion is given (7–9 sq. yds. per gallon), the gritting and rolling being carried out as previously described.

Laying Thickness. The thickness of the carpet is important. If it is too thin, the carpet will possibly scale off. On the other hand, if the thickness is too great, the carpeting material may creep under the action of traffic, leading to the formation of waves or corrugations. A useful general rule is to lay the material to an unconsolidated thickness of at least twice the size of the biggest aggregate used. Thus if $\frac{3}{4}$-in. aggregate is used, the carpet would be laid $1\frac{1}{2}$ in. loose, which would roll down to about $1\frac{1}{8}$ in. The maximum unconsolidated thickness should not exceed three times the size of the biggest aggregate used.

Rolling. Adequate rolling is also important, but the use of a roller which is too heavy or unbalanced should be avoided. A good weight lies between 8–10 tons and wide rear wheels will ensure an even distribution of weight.

General. Limestone has been very successfully used as an aggregate for all types of carpets, either in the form of proprietary materials or in the form of ordinary bituminous or tarred toppings. Fig. 20 illustrates a $\frac{3}{4}$-in. graded bituminous limestone carpet laid in 1938 on A.38. The photograph taken at the end of 1944 shows that this section is still in good condition and an inspection after six years' life revealed that the surface, which has not been treated in any way to date, was still in good condition and definitely non-skid. No signs of corrugation, waviness, or softenings were apparent. The aggregate used was a good Carboniferous mountain limestone coated with a tar-oil fluxed bitumen.

CHAPTER VIII

HOT-PROCESS ASPHALT

Rolled Asphalt (Hot Process)—Mastic Asphalt—Spreading and Consolidation—Summary

UNDER this heading it is proposed to deal with materials specified by B.S. 594 : 1945 under the heading " Rolled Asphalt (Hot Process) ", and with " Mastic Asphalt " which is covered by B.S. 596 : 1945 and B.S. 597 : 1935.

The essential differences between asphalt surfacings of this kind and the bituminous mixtures described in earlier chapters are the very low void content of the mixture as laid, and the use of asphaltic cements of relatively low penetration and in greater proportion.

For example, the specification requirements of B.S. 594 [1] for single-coat asphalt, fine mixture, can be:

Soluble bitumen (30 to 80 pen.)	8·5 per cent.
Passing 200 mesh	9·7 ,, ,,
Retained on 200 mesh, passing 7 mesh	56·8 ,, ,,
Retained on 7 mesh	25·0 ,, ,,
	100·0 ,, ,,

(*Note.*—The largest fragment of stone shall not exceed three-quarters of the final thickness of the consolidated course.)

The above binder content is approximately equivalent to 20 gallons per ton, which is nearly double the gallonage which would be employed with a $\frac{3}{4}$-in. down cold-lay bituminous carpet. The filler content is also proportionately higher. The voids in the mixed aggregate and filler of a well-designed asphalt mixture can be as low as 12·8 per cent.,[2] which means that after the asphaltic bitumen has been added and the material has been consolidated, a very low void content will be attained of the order of 4 per cent. or less.

On account of these three factors, very high mixing and laying temperatures have to be used. For example, in the case of rolled asphalt, B.S. 594 specifies stone temperatures of 250°–350° F. (120°–175° C.) and a minimum laying temperature of 230° F. (110° C.).

The manufacture of hot-process asphalt cannot be undertaken

[1] British Standard Specification for Rolled Asphalt. Fluxed lake asphalt and asphaltic bitumen. Hot Process. (British Standards Institution.)

[2] P. Spielman and A. C. Hughes, *Asphalt Roads*, p. 148. (Arnold.)

on an ordinary tarmacadam plant, and the usual practice is to employ special plants capable of giving the requisite temperatures and to locate these as near as possible to the site of laying. In other words, the drawback that the radius of delivery of these materials is restricted, is usually overcome by sending the aggregate in the uncoated state as near as possible to the site before coating. This is contrary to the usual practice with tar or bituminous macadam, where manufacture is usually carried out in permanent plants at the source of supply of the aggregate, such as the quarry.

Rolled Asphalt, Hot Process. A great diversity of technique exists with this class of material, but B.S. 594 has very successfully incorporated most of the accepted forms of asphalt surfacings coming under this category. Limestone is an accepted aggregate in both these specifications, and it is interesting to note that the limestone is specified to have a crushing strength of at least 14,000 lb. per square inch after heating at 500° F. (260° C.) for a period of 30 minutes. This figure would appear to be unnecessarily low, particularly for a type of surfacing which is usually employed on roads carrying heavy traffic and a minimum crushing strength of 20,000 lb. per square inch would appear to be more suitable, particularly as there would be no difficulty in achieving such figures if a good Carboniferous mountain limestone is used. The absorption of water of the limestone aggregate is specified as not exceeding 0·7 lb. per cubic foot.

The Specification is divided into two parts. Part I deals with Single Course and Part II with Two Course Rolled Asphalt.

Part I defines two types of mixture:

(*a*) Fine Mixtures (Table 2).

(*b*) Coarse Mixtures (Table 3).

It is interesting to note that each Table shows two Schedules of mixtures. Schedule 1 (Table 2) and Schedule 3 (Table 3) are intended for roads in dry situations and where equable climatic conditions prevail as in Southern England, Schedules 2 and 4 are suitable for use on wet and/or cold situations as in the northern counties of Great Britain and Ireland. The basic differences between these two sets of Schedules in the higher bitumen content of Schedules 2 and 4.

Four alternative stone contents (25 per cent., 30 per cent., 35 per cent., or 40 per cent.) are given for Fine Mixtures, with a pro-

Fig. 21.—Hot Asphalt Limestone Surfacing on A.31 near Alresford. Laid July–August 1930. Photographed June 1945.

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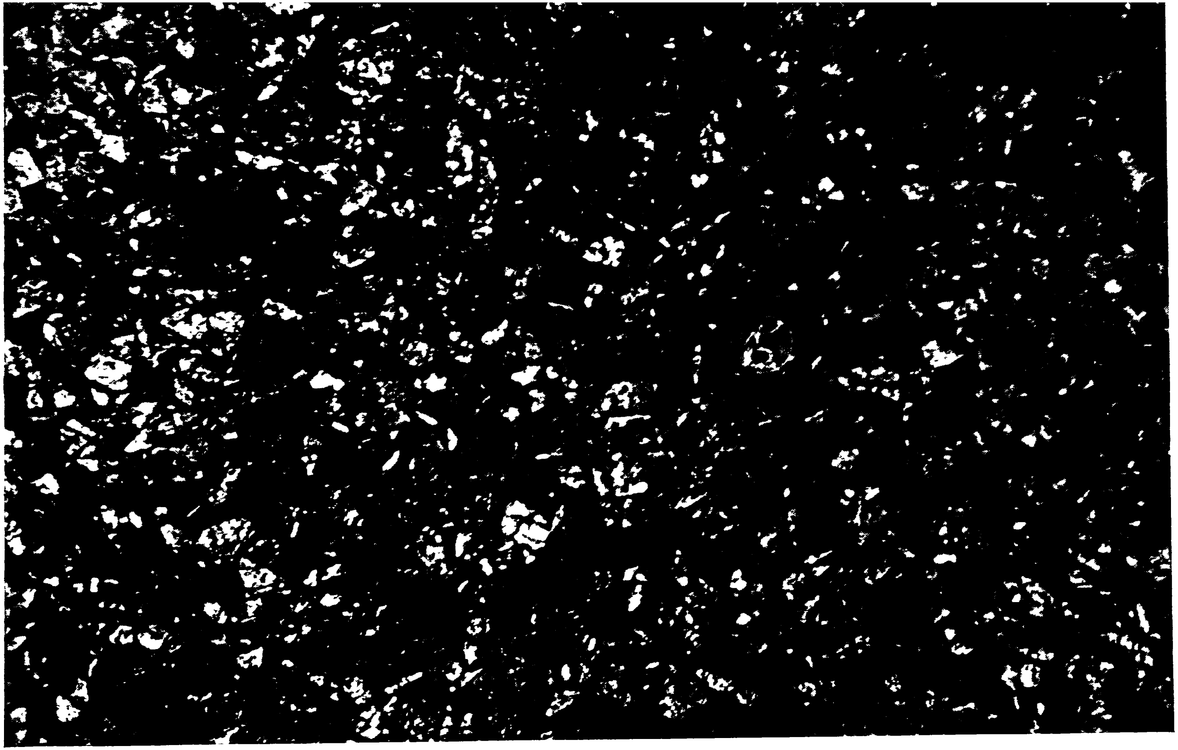

Fig. 22.—Close-up of Surfacing shown under Fig. 21.

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[*Facing p.* 95.

gressive decrease of soluble bitumen specified. The mean soluble bitumen contents are as follows :—

Schedule I. 8·5 per cent., 8 per cent., 7·5 per cent. or 7 per cent.

Schedule II. 9·5 per cent., 9 per cent., 8·5 per cent. or 8 per cent.

In the same way varying stone contents are specified for coarse mixtures, ranging from 45 per cent. to 65 per cent. in steps of 5 per cent. The mean bitumen content varies from 7·5 per cent. to 5·2 per cent. in Schedule 3 and from 8·3 per cent. to 5·8 per cent. in Schedule 4.

Part 2 deals with Two-coarse material. The base course limits are given in Table 2, and the wearing course in Table 3. As in the case of the Single Course materials each Table is sub-divided into two Schedules, the first in each case being recommended for roads in dry situations and where equable climatic conditions prevail.

The stone content of the base course can vary from 55 to 75 per cent. in steps of 5 per cent. with bitumen content variation from 7·15 per cent. to 4·5 per cent. in Schedule I and from 7·7 per cent. to 4·9 per cent. in Schedule 2.

Provision is made for two types of surface in Table 3, which is reproduced in condensed form below.

Type of Surface	Stone (passing ¾ in. ret. ⅜ in.) per cent. by weight	Mean soluble bitumen content, per cent. by weight: Schedule 3 (dry and equable conditions)	Mean soluble bitumen content, per cent. by weight: Schedule 4 (wet, cold conditions)
Sand Carpet	0	11	12
Sand and Stone	15	9·5	10·5
	20	9·0	10·0
	25	8·5	9·5

The penetration range of the Asphaltic Cement for all the mixtures in Parts 1 and 2 is 30 to 80 at 25 per cent. (77° F.). The use of ¾ in. or ½ in. pre-coated chippings is optional for all wearing courses.

Excellent results have been obtained using hard Carboniferous mountain limestone as an aggregate for hot-process asphalt surfacings.

Hampshire County Council, for example, have used hard limestone as an aggregate for this type of material over the past twenty years with good results. These hot-asphalt surfacings have been laid on main roads carrying heavy traffic and have stood up well to the normal pre-war traffic and, in many instances, to exceptionally heavy traffic often of an abnormal type. Experience in this county has shown that the limestone asphalt surfacings have stood up to tank traffic better than concrete, so much so, that at road junctions, where tanks turn abruptly, it was adopted in preference to concrete. Even the oldest of these surfacings are definitely non-skid, and Fig. 21 illustrates such an asphalt surfacing which is still in excellent condition, although laid fifteen years ago.

The specification used in Hampshire County Council is reproduced in Table 29 by kind permission of the County Surveyor, Brigadier A. C. Hughes, C.B.E., T.D., B.Sc., M.Inst.C.E.

TABLE 29

Hampshire County Council Standard Single-coat Asphalt Macadam
(Consolidated thickness 2⅛ in.)

Composition of mixture		Percentage	Analysis percentage
Asphaltic Cement :			
Lake asphalt	7·2		
Mexphalt, 45 pen.	1·4	10·1	7·0
Fluxing oil	1·5		
Filler :			
Portland cement		5·0	7·8
Aggregates :			
Local sand		27·0	29·9
Mendip limestone		57·9	55·3
		100·0	100·0

	Min.	Max.
Penetration of asphaltic cement at 77° F.	65	70

Grading of Mendip Limestone :

	Percentage
Passing ¼ in. B.S.S. sieve	5·0
,, ⅜ ,, ,, ,,	15·0
,, ½ ,, ,, ,,	23·0
,, ¾ ,, ,, ,,	29·0
,, 1 ,, ,, ,,	28·0
	100·0

Grading of Sand (Blended):

	Min. percentage	Max. percentage
Passing 85 B.S.S. mesh, ret. 200 B.S.S. mesh	26	44
" 36 " " " 85 " "	30	50
" 8 " " " 36 " "	19	31

(In accordance with B.S.S. 595 : 1935 and 1152 : 1944.)

Mixing Temperatures:

	Min.	Max.
Mendip limestone	300° F.	350° F.
Local sand	300° F.	350° F.
Asphaltic cement	310° F.	350° F.
Mixed load	290° F.	330° F.

The manufacture of rolled asphalt is highly specialised, and those interested in the theory and manufacture of this type of material will find a great deal of valuable information in the following text-books:

Asphalt and Allied Substances, by H. Abraham; *The Modern Asphalt Pavement*, by C. Richardson, and a more recent work, *Asphalt Roads*, by P. E. Spielman and A. C. Hughes. The testing of asphalt materials is dealt with in *The Testing of Bituminous Mixtures*, by C. Broome.

Mastic Asphalt. This material may be regarded [1] as a material similar in type and grading to that of rock asphalt, but containing a sufficient quantity of bitumen to over-fill the voids in the material and thus enable it to be poured when hot and to spread.

Natural rock asphalt is defined by the British Standards Institution as follows:

"The aggregate shall consist of natural asphalt rock, this term being interpreted in a strictly geological sense and denoting a naturally occurring, consolidated, calcareous rock impregnated with bitumen exclusively by a natural process; the term shall exclude all other types of rock such as result from sporadic bitumen in any mode of occurrence in non-bituminous limestone or in a limestone of low-grade impregnation, non-bituminous limestone and artificial mixtures therefore with bitumen of any source and description." [2]

Two main types of mastic asphalt are standardised by B.S. 596 : 1945 as follows:

(1) Mastic asphalt surfacing with limestone aggregate.
(2) Mastic asphalt surfacing with natural rock asphalt aggregate.

[1] P. E. Spielman and A. C. Hughes, *Asphalt Roads*. (Arnold.)
[2] Reproduced from B.S. 596, by permission of the British Standards Institution, from whom official copies of the specification may be obtained, price 2*s*. post free.

The first type is sometimes described for convenience as "Synthetic Mastic".

Mastic Asphalt (Limestone Aggregate). Under Part 2 of B.S. 596 dealing with this type of material, it will be noted that only limestone or siliceous limestone containing not less than 70 per cent. calcium carbonate is specified. The grading requirements of the aggregate are as follows:

TABLE 30

	Percentage by weight Min.	Max.
Passing 200-mesh sieve	30	50
Passing 100-mesh sieve and retained on 200-mesh sieve	10	20
Passing 25-mesh sieve and retained on 100-mesh sieve	15	30
Passing 7-mesh sieve and retained on 25-mesh sieve	0	25

Reproduced from B.S. 596 by permission of the British Standards Institution, from whom official copies of the specification may be obtained, price 2s. post free.

This aggregate is incorporated with the necessary percentage of asphaltic cement at a temperature of from 350° F. to 400° F., and it is specified that mixing shall be carried out for a total time of at least five hours and for not less than two hours after the last portion of the aggregate has been used.

At this stage the composition of the mastic after casting into blocks is specified as follows:

TABLE 31

	Percentage by weight Min.	Max.
Soluble bitumen	14	17
Passing 200-mesh sieve.	25	45
Passing 100-mesh sieve but retained on 200-mesh sieve	8	16
Passing 25-mesh sieve but retained on 100-mesh sieve	12	20
Passing 7-mesh sieve but retained on 25-mesh sieve	0	20

Reproduced from B.S. 596 by permission of the British Standards Institution from whom official copies of the specifications may be obtained, price 2s. each, post free.

Provision is made in this specification that during the process of melting prior to application, ¼-in. to ¾-in. gauge chippings, consisting of clean, hard, igneous, or calcareous rock, angular but not flaky, free from dust and of approved crushing strength and quality, shall be added. The quantity of coarse aggregate added must not be less than 30 per cent. nor more than 50 per cent. by weight of the final mixture. On footways, the maximum size of added chippings can be reduced to ¼ in. but at least 75 per cent. must be retained 25 mesh.

Natural Rock Mastic. The essential ingredient of natural rock mastic asphalt is a limestone impregnated naturally with not less than 6–10 per cent. of bitumen by weight, to which Asphaltic Cement is added so that the ultimate analysis of the mastic shows on analysis 14 to 16 per cent. of soluble bitumen.

The penetration of bitumen employed in the preparation of mastic asphalt is lower than that foreseen for rolled asphalt, and British Standard specification penetration limits are 25 to 50 at 77° F. (25° C.). The same choice of asphaltic cement as given for rolled asphalt is specified in the case of mastic. B.S. 596 foresees the alternative use of refined lake asphalt or asphaltic bitumen, or equal proportions of both. An interesting development of mastic asphalt during the war has been the substitution of a coal-tar pitch for asphaltic bitumen.

During the war a great number of new specifications dealing with mastic asphalt were issued by the British Standards Institution. Although they do not all deal with road-surfacing or paving materials, a complete list is given below for easy reference:

988 : 1941. Mastic asphalt for roofing, type A (limestone aggregate).
1076 : 1942. Mastic asphalt for flooring.
1097 : 1943. Mastic asphalt for damp-proof courses and tanking.
1162 : 1944. Mastic asphalt for roofing, natural rock with high bitumen content (6–10 per cent.).

Spreading and Consolidation. The laying of rolled asphalt requires a considerable degree of skill and experience on the part of the operatives. Spreading has to be rapidly effected so that rolling takes place before the material has cooled appreciably. Mastic does not require consolidation, but here again, the spreading of the plastic mastic is a highly skilled task.

It is now common practice to roll pre-coated clippings into the surface of both rolled asphalt and mastic asphalt surfacings while the material is still warm and in a plastic condition. The British Standards covering these materials all provide for an application of a layer of approved clean ¾ in. or ½ in. hard chippings.

Summary. These types of surfacings carry a high initial cost which must be offset by a long life to be economical.

A hard carboniferous limestone has been proved in practice to meet all the usual requirements of an aggregate for rolled asphalt surfacings.

As far as synthetic mastic is concerned, there is virtually no alternative, as suitable ground limestone has been found, after many years of experience, to be the only satisfactory material as the aggregate for the manufacture of this type of mastic.

CHAPTER IX

CONCRETE ROADS

by J. SINGLETON-GREEN, M.Sc., A.M.I.Mech.E., M.I.Struct.E.

Slipperiness—Compressive Strength—Flexural Strength—Watertight Concrete—Durability—Correspondence—Examples

Slipperiness. An objection sometimes raised to the use of limestone is that it forms a slippery surface. This argument is probably the result of a statement made by someone who has seen the effect caused by a very soft limestone or chalk. Such an aggregate cannot be compared with a structurally sound limestone, and all the available evidence points to the fact that a good limestone concrete is no more slippery than concrete made with any other aggregate. If a limestone concrete is found to be slippery, it may be that this state of affairs has been brought about by having the surface finish too smooth, and it is a question of the making of the concrete rather than one of the aggregate used.

It may be mentioned [1] that very few concrete roads having an unsatisfactory resistance to skidding have been brought to the notice of the Ministry of Transport, and that in each of these cases it has been found that the surface of the road contained an unduly high proportion of sand and cement and a correspondingly low proportion of coarse aggregate. This condition probably results from excessive tamping given at the time of construction in order to ensure that the concrete should be dense and its surface free from irregularities. The fine texture due to the presence of too much cement and sand in the surface renders the concrete liable to wear smooth under traffic.

Compressive Strength. As the compressive strength of concrete is still, in a large number of cases, the property by which concrete quality is measured, it is necessary to have sufficient strength to meet all specifications. There is no difficulty at all when limestone is used as the aggregate. The results which follow, although not dealing with road construction, prove very conclusively that compressive strengths of a very high order are obtainable with limestone concrete.

[1] *Experimental Work on Roads*, Report for 1938–39 of the Experimental Work on Highways (Technical) Committee. (H.M.S.O.)

Viaduct at Hope. The whole of the concrete in the reinforced portion of Edale Viaduct was designed for a working stress of 750 lb. per square inch. The mixture adopted was 3½ parts crushed limestone ¾ in. to ⅛ in., 1¾ parts Whaley Bridge sand, and 1 part cement, the concrete being mixed as stiffly as possible for the work in hand. The slump for the heavier reinforced concrete members was 2 in., and for the lighter, or more heavily reinforced, members not more than 4 in.

Test cubes were made frequently from the concrete used, and the average compression results in lb. per square inch were:

	7 days	28 days	90 days
2-in. slump	3,707	5,490	6,240
4-in. slump	2,110	4,260	5,100

The following were the factors of safety at different ages:

	7 days	28 days	90 days
2-in. slump	4·9	7·3	8·3
4-in. slump	2·8	5·7	6·8

Mass concrete foundations were placed under the reinforced concrete footings for the piers, the mixture for these being:

10 parts crushed limestone 2½ in. to ⅛ in.
3 parts crushed limestone ¾ in. to ⅛ in.
2 parts Whaley Bridge sand
1 part cement

The strengths obtained with this 15 to 1 concrete, in lb. per square inch, were:

	7 days	28 days
1-in. slump	1,390	2,470

Concrete Kerbs. In October, 1931, an examination was made of limestone in the following grades: ¾ in., ½ in., ⅜ in., dust. The dust was of good, coarse quality and fairly well graded, but contained 18 per cent. passing the 180-mesh sieve. As the aggregates were required for kerb making, the following mixture was used for compression tests: 1 part ¾ in., 1 part ½ in., 1 part ⅜ in., ½ part dust, and 1 part rapid hardening cement, with the following results:

7 days: 4,725 lb. per square inch
28 days: 6,565 „ „ „ „

The kerb made with this mixture weighed 143 lb. per cubic foot.

7,000-lb. Concrete. Three samples of crushed limestone were examined in April, 1929, i.e., ¾ in.–⅜ in., ⅜ in.–⅛ in., ⅛ in. and down. The sieve analysis of the last sample was as follows:

	Percentage
Retained on 120-mesh sieve	82·7
" " 50 " "	74·3
" " 30 " "	65·5
" " $\frac{3}{64}$ in. mesh sieve	46·3
" " $\frac{3}{32}$ " " "	29·6
" " $\frac{3}{16}$ " " "	1·2
Material washed through 180-mesh sieve . . .	20·0

To get a dense mix the proportions used were 2 parts limestone $\frac{3}{4}$ in.–$\frac{3}{8}$ in., 1 part limestone $\frac{3}{8}$ in.–$\frac{1}{8}$ in., 1 part limestone $\frac{1}{8}$ in. and down, 1 part cement. The slump was 1 in. The following were the results of the cubes.

7 days: 4,155 lb. per square inch
28 days: 7,155 " " " "

Another Example. Tests were made in December, 1933, with 1 in.–$\frac{1}{2}$ in. limestone (fineness modulus, 7·19), sand (fineness modulus 2·33) and cement, the concrete having a 1-in. slump. The following results were obtained:

	Compressive stress lb. per square inch 4 days	7 days	28 days
$3\frac{1}{2}:1\frac{1}{2}:1$	3,920	4,665	6,905
$3\frac{1}{2}:2\ :1$	3,420	4,575	6,035

To confirm these figures, check tests were made, with the following results:

	Compressive stress lb. per square inch 4 days	7 days	28 days
$3\frac{1}{2}:1\frac{1}{2}:1$	4,420	5,290	7,155
$3\frac{1}{2}:2\ :1$	3,735	4,665	6,285

Trench Linings. Some interesting tests were made in April, 1939, to see what results would be obtained with a rather small coarse aggregate. The concrete was composed of $3\frac{1}{2}$ parts limestone $\frac{3}{8}$ in.–$\frac{3}{16}$ in., $1\frac{3}{4}$ parts washed basalt $\frac{1}{8}$ in. and down, and 1 part rapid hardening cement by volume. 6-in. cubes were made and cured in the approved manner, and tested at about 30 days. The detailed results are given below:

Cube reference	14090	14091	14092
Slump in inches	2	$1\frac{1}{2}$	$2\frac{1}{2}$
Age in days when broken	32	30	28
Weight in lbs.	18·2	18·2	18·4
Breaking load in tons	95	101	93
Breaking load in lb. per square inch . .	5,910	6,285	5,785

Comparative Tests. In 1936 comparative tests were made by an independent tester to see if limestone could be considered as good as another aggregate which was recognised as being satisfactory for concrete.

The results are given below:

Mix	Compressive stress lb. per square inch 7 days	28 days
4 cub. ft. limestone chippings (¾ in.– 3/16 in.)	3,937	5,410
2 cub. ft. river sand	4,107	5,215
90 lb. Portland cement	3,985	5,253
Gauged with 6·33 per cent. water		
Average . .	4,009	5,293
4 cub. ft. river shingle (¾ in.– 3/16 in.)	3,565	5,418
2 cub. ft. river sand	3,564	5,214
90 lb. Portland cement	3,430	4,955
Gauged with 6·33 per cent. water		
Average . .	3,519	5,196

Strengths at 28 Days. For convenience, the various strengths obtained in the above tests (6-in. cubes) at 28 days are grouped below:

Proportions by volume	Slump	Compressive stress at 28 days, lb. per square inch
3½ : 1¾ : 1	2 in.	5,490
3½ : 1¾ : 1	4 „	4,260
10 : 3 : 2 : 1	1 „	2,470
1 : 1 : 1 : ½ : 1	Stiff mix	6,565
2 : 1 : 1 : 1	1 in.	7,155
3½ : 1½ : 1	1 „	7,030
3½ : 2 : 1	1 „	6,160
3½ : 1¾ : 1	2½ „	5,785
4 : 2 : 1	Stiff mix	5,293

Comment. These tests covered a wide range of proportions and were made with different limestones in each case. The results obtained, which are typical of those which can be procured with numerous good limestones, should convince any engineer that limestone is perfectly satisfactory as a coarse aggregate for concrete. There is an ample margin of strength, and there would be no difficulty in complying with any modern specification.

To show just what can be done with limestone aggregate, the following recent results are included. In April, 1943, a company of products manufacturers obtained a five-figure result on concrete at 14 days. In January, 1944, a similar result was obtained by another products company at 28 days. The cubes were made from the normal run of concrete—not specially prepared mixes. In each case the concrete was vibrated. To get these strengths at such early ages calls for good workmanship, good cement, and good aggregates. Brief details of the two cases are as follow:

Item	Case 1	Case 2
Cement	Super-rapid-hardening	Rapid-hardening
Age	14 days	28 days
Compressive stress . .	10,140 lb. per sq. in.	10,080 lb. per sq. in.
Mix	$1\frac{1}{2}:1\frac{1}{2}:1\frac{1}{2}:1$	$2:1\frac{1}{2}:1:1$
Job	Hostels	Sleepers

Flexural Strength. In Bulletin 7 [1] of the National Crushed Stone Association, Inc., Goldbeck draws certain conclusions as the result of investigations made to develop a method for the design of concrete having any desired modulus of rupture.

Some of these conclusions are given below:

1. In concrete roads the property of high resistance of cross-bending, as expressed by the term " modulus of rupture ", is of paramount importance.
2. A wide range in modulus of rupture is produced in $1:2:3\frac{1}{2}$ concrete due to the characteristics of the coarse aggregates.
3. There is no definite relation between crushing strength and modulus of rupture of concrete when different coarse aggregates are used. Weak aggregates may produce concrete of high compressive strength but very low beam strength.

Nature of Aggregate. Many useful results have been obtained from flexural tests, but perhaps there is none quite so interesting as those dealing with the nature of the aggregate. Kellermann [2] found that, in general, aggregates having rounded fragments produce concrete of lower flexural strength than aggregates which are composed wholly or in part of crushed fragments. He also found that there is a fairly well-defined relation between certain mineralogical characteristics of the coarse aggregate and the strength of concrete, calcareous aggregates in general giving consistently higher flexural and tensile strength than siliceous aggregates.

These are rather sweeping statements, and later investigations may show that some modifications are necessary. In the meantime, the good results obtained with limestone as the coarse aggregate cannot be ignored.

Water-tight Concrete. As in other types of structural work, concrete for roads should be reasonably watertight.

[1] " Investigations in the Proportioning of Concrete for Highways," by A. T. Goldbeck. Bulletin 7, National Crushed Stone Association, Inc., U.S.A., Sept., 1931.

[2] W. F. Kellermann, " Effect of Type and Gradation of Coarse Aggregate upon the Strength of Concrete ". *Public Roads*, June, 1929.

To prevent the passage of water through concrete it is necessary to have both a watertight material and watertight construction. The concrete itself may be sufficiently impermeable to prevent the passage of water through it, and yet there may be leakage at joints or cracks. Far too often this point is not appreciated.

Percolation Test. In 1936 a percolation test was made on concrete in which the coarse aggregate was limestone. Details taken from R. H. Harry Stanger's report are given below. This test shows conclusively that limestone concrete is satisfactory as far as permeability is concerned.

Composition. 4 cub. ft. limestone chippings $\frac{3}{4}$ in.– $\frac{3}{16}$ in.
2 cub. ft. Ham River sand.
90 lb. Portland cement.
Gauged with $7\frac{1}{2}$ per cent. water.
Dimensions of Specimen. 5 in. diameter (approx.) by 2 in. thick.
Area under Test. 20 sq. in.
Age at Time of Test. 14 days.
Storage Conditions. Kept in damp cupboard until due for testing.

Pressure		Pressure maintained for	Quantity of water percolating through specimen per hour
Per square inch, lb.	Feet, head		
20	46	One hour	Nil
30	69		,,
40	92		,,
50	115		,,
75	172		,,
100	230		,,
125	287		,,
150	345		,,

The pressure of 150 lb. per square inch was maintained for 18 hours, and the total percolation during that time was 3 c.c.

Cracks. Leakage in a concrete structure may be due to cracks. If there is to be no surface treatment the concrete must be free from cracks, and this presupposes, among other things, sound design.

Concrete cracks because internal stresses are set up which are too great for the material to resist. These stresses are produced by:

1. Changes in moisture content.
2. Changes in temperature.
3. Loading.
4. Chemical reaction.

Expansion and contraction of concrete during wet and dry storage are influenced greatly by variations in the type of mineral aggregate. Sandstone, trap, and gravel concretes may be expected to undergo volume changes 50 to 100 per cent. greater than limestone concrete subjected to the same conditions. However, a given aggregate which produces high expansion in concrete during wet storage may not produce high contraction in dry storage, and vice versa.

Comment.

These notes demonstrate that:

1. Curing conditions are more important than variations in the aggregate, as far as permeability is concerned.
2. Leakage may occur at joints or cracks, even when the concrete itself is permeable.
3. The risk of cracks can be reduced by the use of limestone aggregate.
4. Limestone concrete is eminently suitable for the construction of watertight structures.

Durability. The durability of limestone concrete is most satisfactory, and evidence can be produced to convince the most sceptical that there is no risk at all in using limestone on account of its chemical composition. Structures of all types can be found to show that concrete made with limestone is quite as durable as concrete made with other aggregates, provided the usual precautions are taken in connection with grading, mixing, placing, curing, etc. In other words, with good concrete practice a limestone aggregate will produce first-class and durable concrete.

Volume Change. Durability, or resistance to the action of weather, is a property difficult to evaluate, but it seems reasonable to assume that, other things being equal, the concrete having the least volume change will generally be most durable [1] since it is least subject to internal stresses and possible cracking.

Volume changes accompanying variations in temperature and moisture induce local or internal stresses by themselves in any unrestrained member and may also cause other stresses in a restrained member or in connecting members, due to such restraint. In this respect, limestone has an advantage, as explained under " Durability ".

[1] R. E. Davis and J. W. Kelly, " Volume Changes of Concrete ". *Report on Significance of Tests of Concrete Aggregates.* American Society for Testing Materials, 1938.

It is clear from the above notes that when discussing the question of durability we must not lose our sense of proportion. The most important thing of all is to make concrete which is watertight—everywhere. Given watertight concrete, i.e., concrete which is, and which remains, watertight, at all points and under all normal conditions, there will not be much to fear. Further investigation of this problem, therefore, becomes a matter of considering watertight concrete.

In the meantime, we can leave the argument and investigate the durability of limestone concrete in existing structures.

Correspondence. In addition to the foregoing information on the durability of limestone, the writer has had correspondence with various engineers, and the following extracts from their replies will be of interest:

Crushed Stone Association, U.S.A. (10.11.37). Limestone is universally used in this country for highway construction, that is, limestone concrete construction, and limestone concrete highways are notably free from cracking as compared with other aggregates.

No distinction is ever made in our national specifications regarding the use of limestone in concrete for any purpose, except possibly in connection with sewer work, where limestone of extra high quality is usually specified, together with other aggregates.

Portland Cement Association, U.S.A. (21.1.38). In this country the use of limestone for crushed aggregate as an alternative to gravel is practically taken for granted. Thousands of structures and miles of concrete highways have been built with limestone as coarse aggregate.

Of course it is essential that the limestone be of good quality. Limestone containing cherty materials or considerable argillaceous material of high absorption would be objectionable.

I am attaching a reprint from *Engineering News-Record* discussing the conditions of the Tunkhannock Viaduct after 17 years' service. This indicates that where a good grade of limestone is used, and observing the basic principles of concrete-making, entirely satisfactory results can be secured.

Another project is Wacker Drive in Chicago, built about 14 years ago. This is a double-deck street along the river front extending for a distance of three-quarters of a mile, and in which limestone was used throughout. The concrete is in excellent condition.

Examples. No matter how many arguments are put forward to show that a particular kind of concrete is satisfactory, the practical man will not be satisfied until he can see the results of actual tests on the structures concerned. The reports given below, therefore, on three experimental lengths of road are both interesting and enlightening.

Gloucester-Newport Road. Experimental concrete sections [1] were constructed in June, 1932. Sections were laid to compare concrete made with Frampton gravel, Clee Hill granite, Malvern granite, and local mountain limestone, the slabs being in each case 9 in. thick and reinforced with a single layer of high-tensile steel weighing 5½ lb. per square yard.

The Frampton gravel was mixed with cement in the proportion of 4½ : 1, whilst the mixes used on the other three sections contained 3 parts of aggregate, 1½ parts of Holm sand, and 1 part of cement. All four sections are in excellent condition and no difference is to be observed in their behaviour.

All the aggregates [2] are capable of providing a satisfactory running surface.

Dock Street By-Pass, Newport, Mon. This experiment was undertaken in 1934 [3] in view of the freedom from cracking of two-course 9-in. unreinforced concrete laid in the city of Cardiff. In this work limestone up to 2-in. gauge is used in the top course, and the slabs are 12 ft. long. It was decided to investigate whether the freedom from cracking was attributable to (*a*) the length of the slabs or (*b*) the use of large aggregate in the surface.

The road was inspected in March, 1937, when all three sections were in good condition generally. The running surface was excellent, but slight spalling was noticed at some of the joints. All the sections have so far given quite satisfactory results without cracking.

The number of sections [4] included in the experiment is not sufficient to enable the influence of all the variables involved to be determined, but the results show that if unreinforced concrete is laid in 12-ft. slabs, a comparatively cheap construction (viz., single-course 9-in. concrete of a 4 : 2 : 1 mix, with dolomitic limestone of either ¾-in. or 2-in. maximum gauge) can be used without cracking.

The freedom from cracking in the 9-in. concrete in 30-ft. slabs using dolomitic limestone aggregate is interesting as compared with the extensive cracking of the 8-in. concrete in 30-ft. slabs using ¾-in. gravel aggregate at Harmondsworth.

[1] *Experimental Work on Roads*, Report for 1936–37 of the Experimental Work on Highways (Technical) Committee. (H.M.S.O.)
[2] *Ibid.*, 1937–38. [3] *Ibid.*, 1936–37. [4] *Ibid.*, 1937–38.

Maghull Diversion, Lancashire. The road has been inspected [1] at intervals, the latest inspection being made in April, 1937. The surface of the concrete was uncracked and in excellent condition. The only section which could be said to differ from the others was Section L, in which limestone aggregate was used, which was slightly lighter in colour.

One of the objects [2] was to compare the relative merits of single-course work and two-course work with cheaper aggregate in the bottom than in the top course. The results of this experiment are in agreement with those obtained elsewhere, viz.:

> There is no advantage to be gained by using two-course instead of single-course concrete.
>
> Satisfactory results can be obtained with a variety of aggregates, including granite, limestone, and gravel.
>
> A 4 : 2 : 1 mix in the running surface gives satisfactory results.
>
> There is no justification in ordinary circumstances for a more expensive surfacing than concrete 8 in. thick, 4 : 2 : 1 mix, doubly reinforced.

[1] *Experimental Work on Roads*, Report for 1937–38 of the Experimental Work on Highways (Technical) Committee. (H.M.S.O.)

[2] *Ibid.*

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