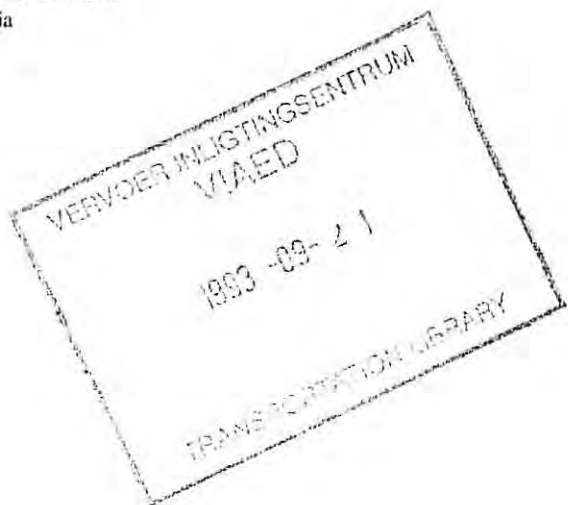# THE NATURAL
ROAD CONSTRUCTION MATERIALS
OF SOUTHERN AFRICA

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ACADEMICA / PRETORIA / CAPE TOWN

FOREWORD

Naturally occurring materials, such as rock and soil, constitute a significant portion of the materials content and cost of a road. Most textbooks dealing with road construction materials, however, are mainly concerned with the manufactured materials used in such structures, such as bitumen and cement. Furthermore, where reference is made to natural materials in such publications it applies mainly to overseas countries.

Since 1957 the author and other researchers in the National Institute for Transport and Road Research have devoted much effort to understanding the properties and performance of natural road construction materials in the variable Southern African conditions. It has been found that different types of rock perform differently in the various layers of a pavement and that these differences are greater in rocks than in soils. It has also been found that environmental conditions, such as climate and topography, play an important role in the performance of such materials in roads. Consequently the durability of natural materials is an important factor in their selection for use in roads.

These findings have been documented in more than 50 publications, many of them in foreign journals and conference proceedings, which cannot always be easily traced. The author has therefore undertaken the task of summarising all the information in this book which will undoubtedly serve as a valuable work of reference for all those concerned with the planning, design, construction and maintenance of roads. Although the book was written primarily for the benefit of Southern African engineers and engineering geologists concerned with roads, much of the information will be of value in other fields of civil engineering both in Southern Africa and in other parts of the world, particularly those with similar geological and environmental conditions.

The reader's attention is drawn to two problems of denomination:

1. The names 'systems', 'series' etc. of the South African stratigraphical hierarchy are going out of use, being replaced by 'supergroup', 'group' etc. in conjunction with certain other stratigraphical changes. At the time of completion of this book, however, the changes were not yet final and, therefore, the old stratigraphical names as shown on the 1970 edition of the Geological Map of South Africa and the Kingdoms of Lesotho and Swaziland, 1: 1 000 000, have been used.

2. All territorial names given in this book are those which were officially valid at the end of 1978.

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CHAPTER I
INTRODUCTION

ANCIENT ROADS

Wherever man has lived or still lives, there is a need for easy and rapid movement across the countryside. The development, or more or less deliberate creation, of the means of such movement has always been unavoidable. Originally there were nothing more than tracks to the feeding or drinking places of ancient man, and these hardly differed from the tracks made by any other mammal for the same purpose. Such tracks were not used for making contact with other groups of men. In fact, deliberately created lines of communication between different groups of people appeared rather late in the history of man. Although the first tracks were somewhat fortuitous, certain improvements to them were probably made quite soon, like the removal of obstructions such as a rock, a tree or a group of thorny bush, to improve the speed of movement and, if necessary, of running away.

Lines of communication which were more than simply tracks, probably did not appear until the increasing number of people in certain parts of the world and changes in the social structure and organization of communities made repeated or even permanent contact between such communities unavoidable. These lines of communication could now be termed roads, since they were prepared for use by men and animals carrying loads and, after the invention of the wheel in about 3 500 B.C., chariots and wagons. The first hard roads are known to have existed in Mesopotamia by the time of the invention of the wheel, or shortly afterwards, and a stone-paved road built before 1 500 B.C. was found in Crete.

These ancient roads served for the transport of merchandise and, perhaps of even greater importance, for the fast movement of military contingents which were beginning to increase in number and in the weight of their armour. In fact, the earliest fully documented battle in history was that of Kadesh in 1296 B.C. when Muwatillis, King of the Hittites, defended one of the important roads of that time through Syria against the advancing Ramses II, Pharaoh of Egypt (Ceram, 1955). Kadesh was situated on the banks of the river Orontes, south of the Bahr-et-Homs just north of the present Lebanon-Syria border. Both kings had about 20 000 men and a large number of war-chariots and other vehicles under their command, and to both of them possession of this road, which was an almost direct link between the capitals of these ancient empires, was of vital importance. As a matter of interest it may be mentioned that Muwatillis retained control of Kadesh.

Most of these ancient roads were probably little more than the old tracks straightened out; they were definitely not designed and constructed in the present sense, but local chiefs were probably under an obligation to keep them in order so that they could be used each season. There must therefore have been some kind of maintenance.

This is not to say that fully designed and constructed roads did not exist in ancient times, witness the one on Crete which has already been mentioned. They

were, however, mostly confined to the cities (*Plate 1*). Amongst them was the marvellous Aibur-shabu, the sacred route for the procession of the Great Lord Marduk, the supreme deity of Babylon, from the Illu gate through the Ishtar gate into the city, built by Nebuchadnezzar II in about 600 B.C. This road was constructed on a fill of compacted soil using sun-dried brick for a layer similar to a base. Dressed limestone and breccia were bound to the brick by a 'tack coat' of natural asphalt. The limestone and breccia, whose joints were also filled with asphalt, constituted the surfacing (**Ceram**, 1949). The Aibur-shabu road of Babylon was thus carefully designed and constructed, using what could be called 'manufactured' materials (the bricks) and natural materials (the dressed stone and the asphalt). In most of the trunk roads of these times, however, only the natural materials that were available at or very near the site were employed. During the 6th century B.C. at least one proper highway was constructed through the desert to connect Babylon with Egypt (**Oglesby**, 1975).

ROMAN ROADS

Half a millenium after Nebuchadnezzar II built the Aibur-shabu, the Romans became the great road builders. In fact, the Roman Empire could only be governed and controlled by means of roads which provided for the swift transmission of messages and the fast movement of military contingents; the transport of goods in bulk was largely done by ship. There were famous Roman roads like the Via Appia (*Plate 2*) and the Via Flaminia, but there were many more and much longer roads as well. These Roman roads were masterpieces of road construction, designed and built with great engineering skill. Some were actually so over-designed that parts of them are still in use today (**Oglesby**, 1975). The roads were constructed mainly of soil and dressed rock, bound with a type of mortar which the Romans produced from limestone which was ground and burnt, and from shale, and a siliceous component. This latter component was volcanic ash obtained partly from the tremendous masses of this ash found on the Greek island Santorin, the relic of the volcano Thera which erupted or rather exploded just before 1 200 B.C., and mainly from the vicinity of the village of Pozzuoli, north of Naples, from which the name 'pozzolana' has been derived.

The decline of the Roman Empire was associated with a decline of road construction almost everywhere in Europe lasting for about one and a half milleniums. It was not until the 18th century that road construction revived as an engineering enterprise.

18TH AND 19TH CENTURY ROADS

The new rise of road construction sprang from the improvement of the earth roads, which had prevailed during the preceding centuries and were mostly hardly more than improved tracks (**Collins** and **Hart**, 1947). Merchandise and military movements again provided the principal momentum behind the new develop-

ments. Prominent in these new developments were people like the French engineer Trésaguet (1716-1796) without whom Napoleon's system of roads, again mainly motivated by military needs, would not have been possible, and the British road builder Macadam (1756-1836) whose name is still connected with a type of surfacing.

From the 18th to the early 20th century roads were predominantly gravel roads and, when they possessed a permanent surface, the carriageway was mostly plastered with dressed stone or cobbles (*Plate 3*). These roads were built almost exclusively of soil, rock and water, i.e. natural materials, and manufactured materials, especially cementitious ones, played a very insignificant role. In general, the development of road construction was still slow, with goods in bulk transported mostly on waterways and, from the middle of the 19th century, by rail (**Collins** and **Hart**, 1947; **Oglesby**, 1975).

MODERN ROADS

Road construction as it is understood today is just over half-a-century old, and is a consequence of the advent of the automobile. During these five decades the modern road, built to carry heavy loads, developed. It now consists of several well-defined layers of pavement (nowadays mostly stabilized with some cementitious material although water-bound design has not been entirely discarded), and has a weather-proof, smooth surfacing (*Plate 4*). Many of the modern improvements have been brought about by the development and introduction of manufactured materials such as cement, lime, bitumen and tar, but in spite of these, natural materials are still of such importance that usually considerably more than half of the costs of a road project go into the selection, recovery, testing and use of these materials.

Modern road construction developed in Europe and in North America, i.e. in parts of the world with similar geological and climatic conditions. The period of development of modern roads is, however, also the period of the major European impact on the world in general. The provision of lines of communication is always the first prerequisite for opening up a virgin area and, since roads can be built faster than railways especially if one is satisfied with gravel roads for a start, road builders in particular provide the first means of movement and transport. The road builders, most of whom initially came to new countries from Europe and occasionally from America, brought with them experience from their home countries. They found rock, gravel and soil which often looked like that known to them at home. They therefore called the materials by the same names they had used at home and treated them accordingly. They built roads with these materials, often in an environment which was unfamiliar to them also using the manufactured materials such as cement, lime, bitumen or tar, and they either succeeded or failed. They discovered then that their previous experience was not always fully applicable to the different environment they now encountered, just as methods used and experiences gained elsewhere, which they read about, could not always be transferred unmodified to all other regions of the Earth.

During the last five decades the fast-growing road networks almost everywhere in the world have made road construction an industry in its own right, involving large investments and the possibility of large gains or losses. Very large sums of money are involved in each road project, and design and construction methods are therefore of paramount importance for the economy of the undertaking. In consequence much effort has gone, and is still going, into the improvement of the methods of design, construction and, more recently, quality control of all types of road. The quantity of published material on these subjects makes it impossible to list all relevant publications and in the following chapters only those which are of direct concern to the problem under discussion have been referred to.

MANUFACTURED MATERIALS

Although the modern road is still constructed predominantly of natural materials, the manufactured materials, such as cement, lime, bitumen or tar, have always received much more attention, probably because they are direct economic assets. The composition and methods of production can be, and have been, changed over and over again until the material meets the demands of any required usage under all environmental conditions. Large investments are connected with the development and production of these materials, and this has again resulted in a wealth of books and other publications which cannot possibly all be listed here; reference to many of them has, however, been made at the appropriate places in the following chapters.

The position of slag is somewhat uncertain in this context. Industrial slags are, of course, 'manufactured' materials but certain phases in the formation of volcanic rocks produce very similar materials. In contrast to that of other manufactured materials, however, the production of slag is incidental to a large extent. Industrial slag therefore combines properties which are akin to those of pumice, volcanic tuff and volcanic ash with a chemical and mineralogical composition which, although it shows some similarity to the composition of basic crystalline rocks, is generally foreign to natural materials.

NATURAL MATERIALS

In spite of the large quantity of natural materials required for the construction of roads, and the large amount of money spent on their selection and use, comparatively little work has been done on them and consequently much less has been published about rocks and soils as road construction materials than about other aspects of road building. It appears that road engineers have mostly accepted natural materials as something like a fate about which not much can be done. Much of this attitude may be related to the way in which the compulsory course in geology, which they had to attend during their pregraduate years, was offered and which created in many of them somewhat of a horror of the subject. They were instructed about rocks and a little about soils; they learned that there are igneous, sedimentary and metamorphic rocks, that each of these genetic groups of rocks could be subdivided again and that the rocks were composed of a multitude of minerals. In their practical work, however, the engineers and engineering geologists dis-

covered that there were suitable and unsuitable engineering materials within each of these genetic groups and that consequently this grouping did not help much in the assessment of aggregate quality: they needed to know the technical possibilities of each individual type of rock. These inconsistencies between the petrological, genetically-based classification and the engineering requirements have already been recognized in standard specifications for fresh rocks issued by the **British Standards Institution** (1954) and the **South African Bureau of Standards** (1976), but even these classifications are still influenced by the traditional genetic grouping.

If the weathering of rocks also has to be taken into account matters become more complicated still. Weathered materials are, of course, of particular concern to everyone dealing with road construction in Southern Africa because, due to the peculiar conditions in the sub-continent, in most cases 'natural gravel' is synonymous with weathered rock. This importance of weathering as an agency for the development of natural gravel is one of the great differences between conditions in Southern Africa and geographically similar parts of the world, and Europe and North America. Weathering may be a simple breaking-down process in certain rocks under all environmental conditions. In other types of rock it may be simple in one environment and very complicated in another. Each time the suitability, and especially the durability, of such materials when used for road construction will be affected differently.

The environmental conditions concerned refer primarily to climate although the topographical setting may have a modifying effect as well. Weathering, with its causes and processes, thus complicates matters further, and the mineralogical and petrological make-up of the rocks, and the environment and its effects on such rocks, must all be taken into account when assessing the quality of natural road construction materials. Previous discussions about the suitability of Southern African rocks for road construction (**Walker**, 1944; **Knight** and **Knight**, 1948) have largely avoided considering the influence of weathering on the engineering properties of natural materials in different parts of the sub-continent.

In contrast to manufactured materials, natural road construction materials cannot be changed at will to suit the purpose; they have to be accepted as they are. Being natural, they are extremely variable: there are numerous transitions from one type to another and, at least under the climatic and topographic conditions of Southern Africa, many of them weather differently in different parts of the sub-continent, so causing the formation of different types of gravel from the same parent rock. It must be realised further that such materials do not only degrade during the construction of a road and under traffic, but that weathering continues after these materials have been laid in a pavement. This continued weathering may cause certain natural materials to change to such an extent that they lose those properties and characteristics which led to their selection. This means that each material may, at the time of its selection, be in a condition which is detrimental to its durability, even if it appears to be acceptable initially.

Since natural road construction materials are more complex than manufactured ones, and since none of them is made to suit a particular purpose, the selection of natural materials has always been based mainly on experience. Testing has been

aimed primarily at the requirements of design, and it has been based on the supposition that the properties of a natural material, once tested and found satisfactory, will not change during the structural design life of the road. The actual performance of natural materials in the road has shown that this supposition is often true, but there have also been cases, often with very costly consequences, where the material failed in spite of satisfactory test results at the time of selection.

The variations in quality, especially in durability, and the underlying reasons for these variations are discussed in the following chapters. This discussion is meant to be a guide for engineers and engineering geologists and confines itself to what is thought to be absolutely necessary. Certain data which often occur in books of this kind but which can be found in other relevant books, have not been included since they would only be 'fillers' and particular points can be made as well without them. Examples of such data are the details of the mineral composition of individual rocks, chemical formulae of minerals and a chemical notation of the changes which occur in a primary mineral decomposing into clay. It has been felt that such data are only of academic interest to engineers while an engineering geologist should know them anyway. Such data are given only where they are thought to be essential or where they are something of a novelty and are not yet readily found in other publications.

An attempt has been made to reduce the complex natural relationships and interactions between rocks and soils in a variable environment to a few basic principles, thus utilizing an empirical approach of seeing complex things in a simple way. In this simplified approach, the presence or absence of quartz in a rock, fresh or at any stage of weathering, intended for use in road construction is used to assess its potential durability under the various Southern African environmental conditions, expressed by the climatic N-value, and modified to a degree by topgraphy, expressed as the angle of slope. Durability cannot be measured in such exact terms as strength, plasticity and other engineering properties: its assessment is an estimate, a calculated guess, which should always be made before embarking on the numerous procedures which eventually lead to a road. Durability should always be the primary requirement when selecting natural road construction materials but, since this property depends on very many independent variables, only general guidance can be given. The successful selection of durable, good natural road construction materials, however, still remains an art rather than a science.

ROCK AND SOIL

GENERAL

Superficially, there appears to be no problem in using the terms 'rock' and 'soil' as they are obviously two distinctly different basic types of material. In everyday communication, 'rock' is the solid, hard material which makes up the actual crust of the Earth while 'soil' refers to the loose, unconsolidated layer, the regolith, which is derived from the break-down, i.e. the weathering, of rocks. This easy and distinct classification is safely applicable as long as only the end-conditions are concerned.

Considering end-conditions, however, involves considering the transitional stages between, and changes from, one end to the other. This in turn means that the boundary between 'rock' and 'soil' must be defined more precisely to describe at what point of the transition the rock changes into soil and vice versa. This then is where the apparently clear distinction between the two entities becomes problematic.

THE SOIL PROFILE

Distinguishing between soil and rock does not pose problems where transported soil has been deposited on bare, unweathered rock as is frequently the case in recently glaciated areas.

'SOIL' IN ENGINEERING AND GEOLOGY — TILL
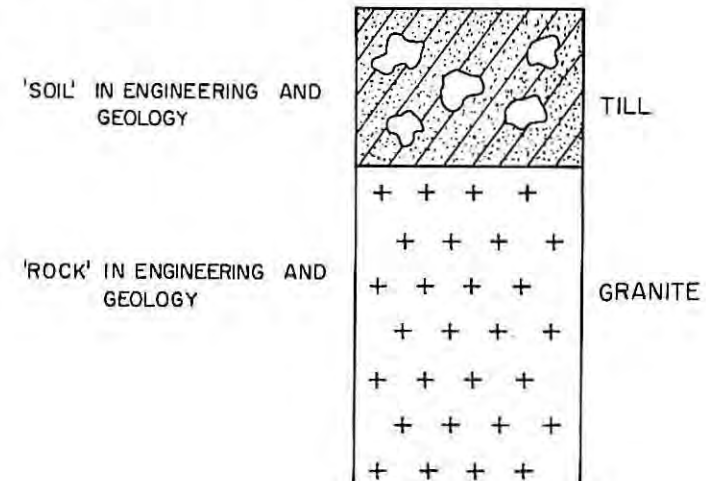
'ROCK' IN ENGINEERING AND GEOLOGY — GRANITE

Figure 1: *Till on unweathered granite*

In Figure 1 glacial till, consisting of an unsorted mixture of boulders, gravel, sand and clay, was laid on fresh granite when the melting and retreating glacier deposited its load of debris. The transported soil, the glacial till, is clearly separated from the underlying rock and there is no transition between the two materials. Consequently, in this situation 'soil' and 'rock' have the same meaning to engineers and geologists.

Such profiles as in Figure 1 are found typically at high altitudes or at high latitudes where weathering, particularly decomposition, is insignificant. The covering soil need not, of course, be only glacial till: it may also have been transported by water or wind, or it may have developed from decomposing plants (*Plate 5*). All such soils may be deposited directly on unweathered rock.

It is not certain whether glacial deposits of the Pleistocene age exist in Southern Africa although their local occurrence cannot be entirely excluded, especially in and around Lesotho (**Butzer**, 1973; **Sparrow**, 1974; **Van Zinderen-Bakker** and **Butzer**, 1973). Most of Southern Africa has been subject to predominantly continental conditions at least since Beaufort times, i.e. over more than 200 million years, and during Cenozoic times the climatic conditions were probably not subject to such radical changes as was the case in large regions at higher latitudes. Under such conditions, weathering and the formation of residual soils have played a very important role.

A typical residual profile as it develops, particularly on decomposing basic crystalline rocks, is shown in Figure 2. In this succession of layers there is no well-defined boundary between rock and soil nor between the different stages of weathering. In a profile like this the underlying, fresh, well-crystallised rock changes gradually into a soil which is composed almost exclusively of clay minerals. Engineers and geologists will have different opinions on the position of the boundary between rock and soil in this profile.

Although residual soils are almost ubiquitous in Southern Africa, transported soils occur hardly less frequently. The thickness of the latter is, however, mostly much less than it is in the recently glaciated parts of the world. Transported soils in Southern African are formed mostly by wind, water, gravity and biotic action. Wind, water and gravity move unconsolidated material more or less laterally, while biotic action results mostly in the vertical transport of soil particles. Selective removal and redeposition according to the mass and shape of the particles are the important results of transport by wind and water. In this way, soils are formed which are very different from the residual soil. Digging insects, mainly ants and termites but also other animals like worms, carry large quantities of soil fines to the surface, thereby creating layers of fine-grained biotic soils. Such soil layers, even if derived from weathered or residual material and deposited on top of it, become transported soils.

Gravity may make unconsolidated material slide downhill. This type of transport, which is usually slow but may also occur spontaneously as a landslide, does not result in much sorting. The resulting soil is colluvium, probably the most widely-occurring transported soil in Southern Africa, and it accumulates mostly on the lower reaches of slopes and in pediments. It may resemble glacial till texturally, the difference being, however, that it is not composed of foreign material but of
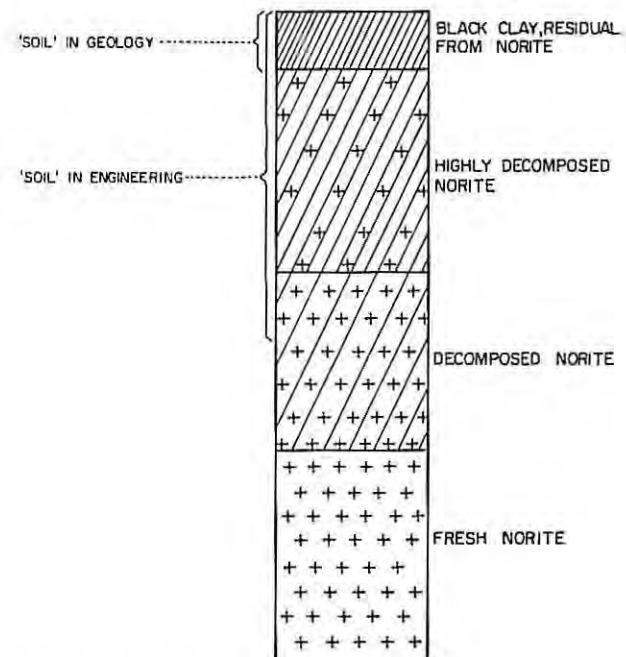

Figure 2: *Profile on decomposing norite*

that occurring just uphill. The composition of colluvium at a site can therefore be predicted more accurately than for instance that of glacial till.

The number of agencies responsible for the formation of transported soils is limited and the type of soil formed by a given agency of transportation depends strongly on the landform. Based on the above, Brink *et al* (**Jennings, Brink** and **Williams**, 1973) have distinguished between the following eight types of topography-related, transported soils with definite compositional and engineering characteristics:

Talus (coarse colluvium)
Hillwash (fine colluvium)
Alluvium (deposits in river plains)
Gully wash (gully and stream linings)
Lacustrine deposits (stream deposits in lakes, etc.)
Estuarine deposits (river mouth and tidal action)
Litoral deposits (wave action)
Aeolian deposits (wind action)

To these may be added the **biotic soils** of which insects and other small animals are the agency of transportation, and which depend very little on landform.

Whenever a mass of soil moves, the heavier particles are inclined to concentrate near the base of such a mass. Stone or gravel layers are thus an indication of some change of conditions which has affected the material above. Such a change could have been the removal or accumulation of material: in both cases a stone or gravel layer is likely to have formed. In the case of removal, this layer consists of the components whose mass was too great to be removed by the force concerned. Such a layer would then form the base upon which subsequent sediment would be deposited, this stone line representing the former soil layer. In the case of the accumulation of transported soil, the heaviest, or rather most massive, components will be deposited first and the finer components will occur on top of the basal stone or gravel layer. Glaciers may be an exception in that they can remove loose material of virtually any mass or size, eventually depositing till on a bare, often polished rock surface. When the glacier melts and retreats, however, interaction between the deposition of till and subglacial erosion caused by melting ice and snow will first create a gravelly or stoney layer in which, eventually, ever-finer material will be caught.

Each such stone or gravel layer represents the basal conglomerate of a new sedimentological process, and it is obvious that several such stone or gravel layers may occur within a sedimentary succession.
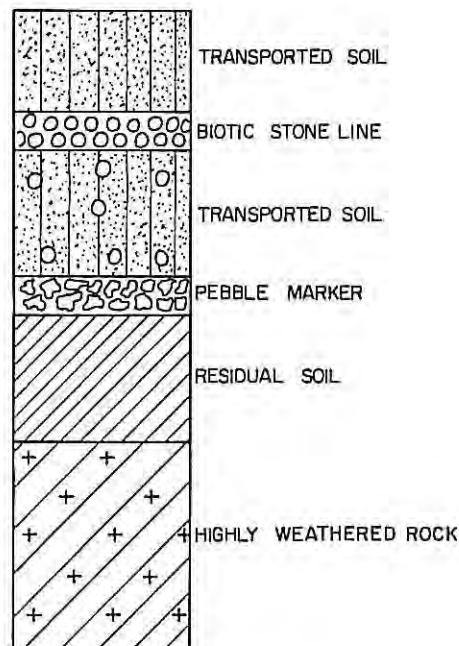

Figure 3: *A biotic stone line and a pebble marker in a soil profile*

In Southern Africa, special significance has been attached to one of these stone or gravel layers because it marks the boundary between residual and overlying transported soil. This layer has become known as the 'pebble marker', a name given to it by **Jennings** and **Brink** (1961). Its presence should be denoted in the description of each soil profile (Figure 3) at least when such a profile intersects transported and residual soil. The pebble marker need not always be present, e.g. a fine-grained wind-blown sand could overlie a black clay that is residual from a basic crystalline rock without a pebble layer between the sand and the clay (**Van der Merwe**, 1967), but in most cases there will be a pebble marker.

Many soil profiles in Southern Africa will reveal the presence of some transported soil, very often in the form of colluvium, resting on residual soil. In cases where the transported soil on top is very different from the residual soil below, as in the example of wind-blown sand on black clay, the distinction between 'transported' and 'residual' poses few problems. In many cases, however, the transported soil is very similar to the residual soil and it may be almost impossible to distinguish between the two if the pebble marker cannot be established. This condition applies particularly to colluvium and to biotic soils.

Colluvial soil may have been subject to so little lateral movement that the colluvial material still rests on the type of residual soil from which it developed. There may thus be very little difference between the colluvium and the underlying parent material. In the course of the slow movement of colluvium, heavier particles are inclined to concentrate near the base of the flowing soil mass and an often ill-defined pebble marker develops.

This similarity between a residual soil and a covering transported soil may be greater still where transported soils develop from biotic action and where vertical transport is much more important than lateral transport. In such cases the soil fines in the transported and residual portions of the profile are the same. The development of a stone line, a biotic stone line, however, is more likely under a biotic soil layer than under any other type of transported soil. The digging insects or other small animals only remove soil fines and carry them to the surface. Larger sand grains and other pieces, too heavy to be carried by these animals, gradually drop downwards where they become concentrated at a level below the surface. This level depends on the depth to which the animals are digging. The biotic stone line which thus develops is, of course, a pebble marker only if it separates transported from residual soil, i.e. if the animals rework a residual soil. *Plate 6* illustrates such a case where a red soil has been formed by biotic action from a red clay soil residual from norite. The norite has been penetrated by quartz veins which disintegrated during weathering. These veins collapsed in the zone of biotic reworking and the pieces of vein quartz moved downward and became concentrated at the lower level of biotic action while the veins in the undisturbed residual soil remained in situ. The biotic stone line so created shows a greater accumulation of vein quartz next to the veins, thinning sideways and merging with the quartz from the next vein. The quartz veins in the lower, residual soil and their absence in the covering transported soil clearly characterize this biotic stone line as a pebble

marker which separates two different types of soil of almost identical composition.

A pebble marker is, of course, not always as easy to recognize as in the above example. Particularly if it is composed of easily weatherable stones, the pebble marker may itself weather and it may then be difficult to detect. This has frequently been observed in the Orange Free State and other predominantly Karoo areas where shale or argillaceous sandstones in pebble markers have weathered so thoroughly that they have almost disappeared. Even in cases of such complete weathering, the former pebble marker can usually be discerned after the face of the outcrop or borehole has been carefully cleaned; although the marker is weathered completely, the weathering will have taken place in situ and the outlines of the former stones are usually sufficiently well-preserved to be recognised in such a cleaned face.

Engineering significance of pebble markers

All stone lines, whether they constitute pebble markers or not, are zones of free drainage. In foundation work, notice must therefore be taken of each stone line. In addition, a pebble marker provides valuable information on the possible availability of construction materials of the desired quality: since the pebble marker separates transported from residual soil, a possible change and improvement in the materials conditions below this particular stone line can be predicted. Such a prediction is possible because from the pebble marker downwards, weathering has been the only soil-forming process and, therefore, fresh rock must occur at some depth below the usually well-known stages of weathering. Moreover, the type of residual soil often enables one to determine the type of rock from which it has been derived. With some experience, it is then possible to estimate the depth at which material of the desired quality may be found.

An exception to these generally valid observations may be found where the weathering rock layer has been so thin that it has weathered completely and no fresh rock has been left. An example of this could be a thin plate of Karoo dolerite; no fresh dolerite may be found in this case and there is only a small chance of finding a relatively thin layer of more or less indurated sedimentary rock.

THE PEDOLOGICAL PROFILE

In pedology, the study of soils with regard to their vegetation-supporting properties, the soil profile is subdivided into three horizons:

The A-horizon: This is the zone of eluviation at the top of the profile from which soluble salts and colloids have been leached and where organic matter has accumulated.

The B-horizon: This is the zone of illuviation which is enriched by materials leached from the A-horizon.

The C-horizon: This is the parent material of the A- and B-horizons; its upper part is still penetrated by roots.

Where these horizons have to be divided further, this is usually done by means of annotations such as A_0, A_1, A_2, B_1, B_2, etc. It may be observed that under Southern African conditions the A-horizon is mostly noticeably more sandy than the B-horizon, and that the accumulation of organic or even humic matter is normally less pronounced than in cooler, more humid climates. The development of these pedological horizons is usually absent in immature soils occurring in Southern Africa mostly, although not solely, in areas where N is more than 10 (see Chapter 3).

The presence or absence of these horizons does not depend on the transported or residual nature of the soil. Horizons may occur in either of them and the pedological classification therefore has no direct bearing on the engineering properties of a soil. The phenomena which govern soil formation in the pedological sense, lead to a much greater subdivision of soils than is required for engineering purposes, and an appropriate simplification will be required when interpreting pedological information or maps for these purposes.

VARIOUS DEFINITIONS

In the extensive literature on civil engineering, pedology, geology and engineering geology, as well as in literature on soil mechanics and rock mechanics, the difference between rock and soil has been discussed by many authors. When their definitions are compared, the only common factor seems to be the apparent impossibility of achieving a definition which covers the issue fully and satisfactorily for all possible cases. Most of these definitions refer to conditions in the relatively cool climates and the, geologically speaking, recently glaciated areas of the northern hemisphere where the quantity of transported soils which are often deposited on an almost unweathered rock base, by far exceeds the quantity of residual soils derived from the in-situ weathering of underlying rock. In these parts of the world the two end-conditions are often in direct contact with each other and there is not much of a transition between them (Figure 1); nevertheless the definitions also vary for these regions. They always include a number of 'provided that's' and 'if's'. Where and on which grounds the distinguishing line is drawn depends greatly on the author's outlook, his special field of study and the purpose for which the definition is being formulated.

A number of authors confine their definitions to either 'rock' or 'soil', thus implying that what is not covered by the definition is the other material. This is not the worst approach to take, but again it does not result in a fully satisfactory description without vague statements or restrictions, even when it is intended for a particular purpose, as shown in the following example: "To the engineer the term rock signifies firm and coherent or consolidated substances that cannot normally be excavated by manual methods alone" (**Department of the Army (USA), 1952**). In this definition, the word 'normally' suggests that there are rocks, e.g. many shales, which can in fact be excavated by hand and a restriction is placed on this otherwise most important and distinctive property of rocks. By simply replacing a few words and retaining 'normally', the above definition would become applicable

to soils, and there would again be an overlap of the areas covered by such definitions.

Dapples' (1959) attempted definition in 'Basic geology for science and engineering', which refers mainly to the USA, may serve as an example of giving a definition of 'soil' without giving one of 'rock'. Basically Dapples defines 'soil' as all the loose material on the surface of the Earth which consists of discrete particles smaller than boulders. Because of the differences of meaning attached to the term soil in various fields of study and where the definition of 'soil' is intended primarily for the use of the soil scientist, Dapples suggests the use of 'sediment' for the product of mere rock disintegration which cannot yet support vegetation. He realises, however, that the term soil has become ingrained in engineering literature to such an extent that this suggestion would hardly meet with the approval of engineers nor, probably, with that of geologists. Moreover, the geologists may argue that 'sediment' has already a much wider meaning in geology.

If the drawing of a satisfactory distinction between rock and soil is hardly possible in parts of the world where complete, gradual transition between them is the exception rather than the rule, how much more difficult would this be in regions where in-situ weathering, disintegration as well as decomposition, is the principal method of soil formation. Such are the conditions in the whole of Southern Africa and only a few South African authors have attempted to define the boundary between rock and soil on the basis of the conditions in the subcontinent.

UNCONFINED COMPRESSIVE STRENGTH

In South Africa two fairly recent attempts to arrive at a distinction between rock and soil were published in the same place within one year, approaching the problem from the angle of soil mechanics in the one case and from that of rock mechanics in the other. In January 1973 **Jennings, Brink** and **Williams** published the 'Revised guide to soil profiling for civil engineering purposes in South Africa' in the Transactions of the South African Institution of Civil Engineers, and in the December 1973 issue of the same Transactions, **Bieniawski's** paper 'Engineering classification of jointed rock masses' appeared. Clearly, from the intention of the papers as indicated by their titles, the authors had to draw some distinction between rock and soil; it is only natural, however, that their limits do not agree. The authors, as others have done before, use the unconfined compressive strength for making the distinction, but the limiting values they give differ; those concerned primarily with soil call a material 'rock' which, in the eyes of those primarily concerned with rocks, is a soil. Consequently there is some overlap: Jennings and his co-authors set the limit between rock and soil at an unconfined compressive strength of 0,7 MPa, while Bieniawski advocates 1 MPa. There is no reason, however, to consider any of the limits set by these authors as 'wrong' because the purpose determines the position of the distinctive point.

There have been more such attempts to define a limit between rock and soil on the grounds of the unconfined compressive strength; the two mentioned above are only the more recent ones in Southern Africa. In their book on road building materials **Knight** and **Knight** (1948), who refer much to South African conditions, avoid the definition of a boundary between rock and soil.

As already stated, many authors confine their definitions to either soil or rock, thus escaping the need to fix such a limit. This type of definition has been given by **Coates** (1964), **Deere** and **Miller** (1966) and **Stepledon** (1968) whose classification of rock, based on the unconfined compressive strength, starts at 2 MPa without stating that everything of a lower unconfined compressive strength must necessarily be a soil. **Broch** and **Franklin** (1972), also employing the unconfined compressive strength for their classification, separate a grade of 'extremely low strength' from one of 'very low strength' at 0,7 MPa, implying that this may also be the boundary between rock and soil. **Broili** (1969) gives grades of weathering, his 'highly weathered' being any material of an unconfined compressive strength of less than 1 MPa and this limit, again set from the point of view of rock mechanics, can be regarded as a suggested boundary between rock and soil as well. In 1970 the Geological Society of London, also working with different grades of unconfined compressive strength, set the limit between rock and soil at 1,2 MPa.

From the engineering point of view, the criterion for the distinction between rock and soil may also be set according to the workability: all materials which are scrapable are considered as 'soil' and all materials which must be ripped or eventually blasted as 'rock'. It is obvious that the strength limit for the distinction between scrapable and rippable is much less than 0,7 MPa and may even be as small as 0,4 to 0,5 MPa. Such a distinction may have a practical basis but is is also not unbiassed since much depends on the type of ripper and, moreover, many materials which are normally described as 'soil', e.g. certain hardpan stages of pedogenic materials, would have to be regarded as 'rock'.

In general, therefore, in a number of attempts to define the limit between rock and soil the unconfined compressive strength has been used as a yardstick and the order of 1 MPa appears to be particularly favoured. Nevertheless, there is still a play of 0,5 MPa or more about the values set by individual authors and, being weathering rocks, many widely-used natural engineering materials in Southern Africa fall into this area.

SEISMIC WAVES

Seismic waves are even less suitable for defining the boundary between rock and soil than is the unconfined compressive strength. The speed of propagation of these waves depends on so many properties of the rock or soil, some of them contributing to the increase and others to the decrease of the rate of wave propagation, that the overlap of such speeds becomes too large to be used for making the required distinction. A well-consolidated, although loose material whose pores are filled with water, e.g. alluvium, may produce wave velocities approaching 1 500 m/s in contrast to porous material, e.g. pumice, the sill phases of lavas, or shattered rock, in which these waves may propagate at less than 1 000 m/s, not to mention the relatively high rate of propagation in certain secondarily cemented materials like calcrete or ferricrete.

SOUTHERN AFRICAN CONDITIONS

Normal contact between rock and soil in Southern Africa is that of the bedrock covered by the weathering residue obtained from this bedrock. There is therefore a continuous transition between the solid rock at some depth and the loose, weathered materials nearer the surface, which are generally known as 'soil' (Figure 2). The exact nature of this soil depends on the type of rock which weathers and the environments in which it occurs. A considerable variation in the properties of these materials is therefore possible. If there is a weathering succession on the bedrock, i.e. if the bedrock does not outcrop or is not covered directly by transported soil, there is no difficulty about recognizing the two end-conditions as 'rock' and 'soil'. However, the question remains where to draw the separating line within the succession.

When a rock starts weathering, cracks occur and the rock breaks up into pieces. These pieces gradually become smaller, being of boulder size first, then pebble, gravel, sand, silt and eventually clay size. This process may cease at any stage, and whatever type of loose material covers the surface could then be called 'soil' if the total mass of this material was considered. Probably, nobody would call a boulder layer 'soil', but from pebble size onwards, nobody would hesitate to do so if the layer as a whole was being referred to, although the individual pebbles in this layer would still be called 'rock'. There would be no doubt about the name 'soil', whether referring to the whole mass or to individual grains, if all grains were below pebble size.

If chemical alteration takes place in addition to the process of physical breakdown, the distinction between rock and soil becomes even more uncertain. The onset of chemical weathering (decomposition) is very similar to the onset of physical weathering (disintegration). At some stage, however, certain rocks may change into a mineralogically different material consisting mostly of sand-, silt- or clay-size grains, while the material as a whole retains the original texture of the rock. Geologically, this stage is still 'rock' but, as regards engineering treatment, it is 'soil'. Only when the residual material eventually loses all traces of the original rock, can it rightly be called 'soil' in both engineering and geology.

Transported soils are not as difficult to classify into rock and soil as are soils in a weathering succession, but although in most cases transported soil is undoubtedly soil, there are deposits where the same uncertainty may arise. Glacial till, volcanic breccia and talus must all be regarded as soil cover if the whole mass is considered, although they may contain many individual pieces which are undoubtedly rock.

Secondary cementation can occur in any loose material, i.e. in any soil. The cementing agents vary but the result is often a material of sufficient hardness and strength to possess all the properties of a rock when used for engineering purposes. Such a hard and strong material, e.g. hardpan calcrete or ferricrete, in spite of its rock-like properties, is soil geologically and pedologically, because it only constitutes a layer within a soil profile. It may or may not be covered by uncemented loose overburden. The actual rock-like layer, whatever its thickness, is usually underlain by uncemented, loose soil. Excavation which makes use of the lower, loose layer, can usually be done by bulldozing, but blasting may occasionally be required as well. The individual blocks of the hard layer, however, must be crushed because they possess all the properties of a rock and must be treated as such for engineering purposes.

South African conditions have been recognized and accounted for in Part 7, 'Standard method of measurement of civil engineering quantities', of the 'Conditions of contract for civil engineering works' (**South African Institution of Civil Engineers**, 1973) and the following definitions have been given:

a) '*rock* is the solid rock occurring in bulk, banks or ledges, or boulders exceeding a specified volume, mostly 0,25 m³, the practical excavation of which normally necessitates the use of explosives;

b) '*hard* is material other than rock whose excavation by pick and shovel would be uneconomic and which requires pneumatic tools or mechanical breakers; and

c) '*soft* is any material not defined by *rock* or *hard*, such as gravel, earth, sand, silt and clay.'

It may be noticed that the term 'soil' is avoided entirely and these definitions do not aim at making a distinction between rock and soil but at describing the conditions which affect excavation. Although the term 'rock' has been retained, the different transitional stages, the end-product 'soil', and any secondary processes which may change the loose soil into some strong material are adequately covered by the descriptive terms 'hard' and 'soft'. A decomposed rock which has to be treated like a soil in engineering works, although it is still a rock in terms of petrology, is now described as 'soft' without entering into an argument about its nature as a rock or a soil. The same applies to those highly cemented materials occurring in calcrete and other pedogenic materials, which are described as 'hard', again indicating only the important condition for excavation and other engineering treatments.

CONCLUSION

There have been many attempts to arrive at a definition of the difference between rock and soil and no proposed definition is really better than any other. Much, if not all, depends on the purpose to which the material is to be put and any of the older definitions, as for example that a 'rock' is a material which can only be sampled using a single-tube core barrel, appears to be as suitable as any other.

The distinction between rock and soil is more difficult to make in geology than in engineering. The geologist must include genetic aspects in his definition and, since soils develop from rock, there are numerous transitional stages which make a clear distinction difficult if not impossible. The engineer is concerned with the characteristics of these materials as they affect the design, construction and performance of his structures. To the engineer, therefore, rock and soil are materials

of different consistency and strength distinguished by differences such as: a rock is solid, its components are grown or cemented together, it is sufficiently strong to be crushed, it fails along predetermined joints or structural planes, it has a high bearing capacity and it does not settle or swell; a soil is not solid, it consists of discrete particles and it cannot be crushed, on failure the shear surface passes through intact material, the bearing capacity varies, and it may settle or swell.

Under conditions like those in Southern Africa where in-situ weathering is the principal mode of soil formation, and where the formation of secondarily cemented layers in a soil profile is frequent, a definition of the difference between rock and soil, which covers all possible conditions satisfactorily, appears even more impossible than in those parts of the world where transported soils, covering a basement of solid rock, occur more commonly than residual soils. Consequently, there are many materials in Southern Africa which are rocks in terms of geology but have to be treated like soils in engineering. There are also materials which are soils in a geological and pedological sense but which, because of the high degree of secondary cementation, require the same engineering treatment as rocks. No attempt will therefore be made here to formulate a new definition. The terms 'rock' and 'soil' will be used in their colloquial sense. Nevertheless, where required, both terms will be specified and explained in as much detail as the description of a condition demands.

ENVIRONMENT

GENERAL

The environment in which each road is built has a greater influence on its life and performance than is often realized. This environment has to be accepted as it is and the design and method of construction must be suitably adapted. The environment of a road is the sum of all those external conditions, inorganic as well as organic, to which a road is exposed and the more the road is in harmony with this environment, the better its performance will be.

During its construction and its service, a road is influenced by climate, the local geology, topography and hydrology, by vegetation and animal life (particularly small species living underground), and by human activities which can have a very strong influence on the pre-existing natural balance.

CLIMATE

The great influence of climate on the quality of natural road building materials, as well as on the general performance of a road, has long been realized. This effect is more pronounced in large countries with great variation in climate, regionally as well as seasonally, and it has therefore been of considerable concern in Southern Africa.

THE N-VALUE

Work on the effect of climate on roads, which eventually led to the N-value, first began when it was observed that weathered basic igneous rocks, Karoo dolerite in particular, performed differently in different parts of the Republic of South Africa. There was a marked difference in the durability of these rocks when they were used in the base and other layers of the pavement in the eastern and the western parts of the country. Road engineers in Natal, the Transvaal, most of the Orange Free State and the eastern parts of the Cape Province (Transkei) were greatly concerned about the performance of these materials which were almost the only natural materials worth considering in most of these areas; road engineers in the western parts of the Republic, on the other hand, were generally satisfied with the use of weathered Karoo dolerite in the structural layers of a pavement. The known boundary between generally satisfactory and unsatisfactory performance ran from Port Elizabeth through Bloemfontein to Mafeking.

Such differences in performance were particularly noticeable where weathered dolerite had been used for the base of bitumen-surfaced roads. Such roads in most of the Cape Province and in the western parts of the Orange Free State gave satisfactory service for more than twenty years. But in the eastern parts of the Republic, i.e. east of the above boundary, severe cracking and loss of shape occurred in
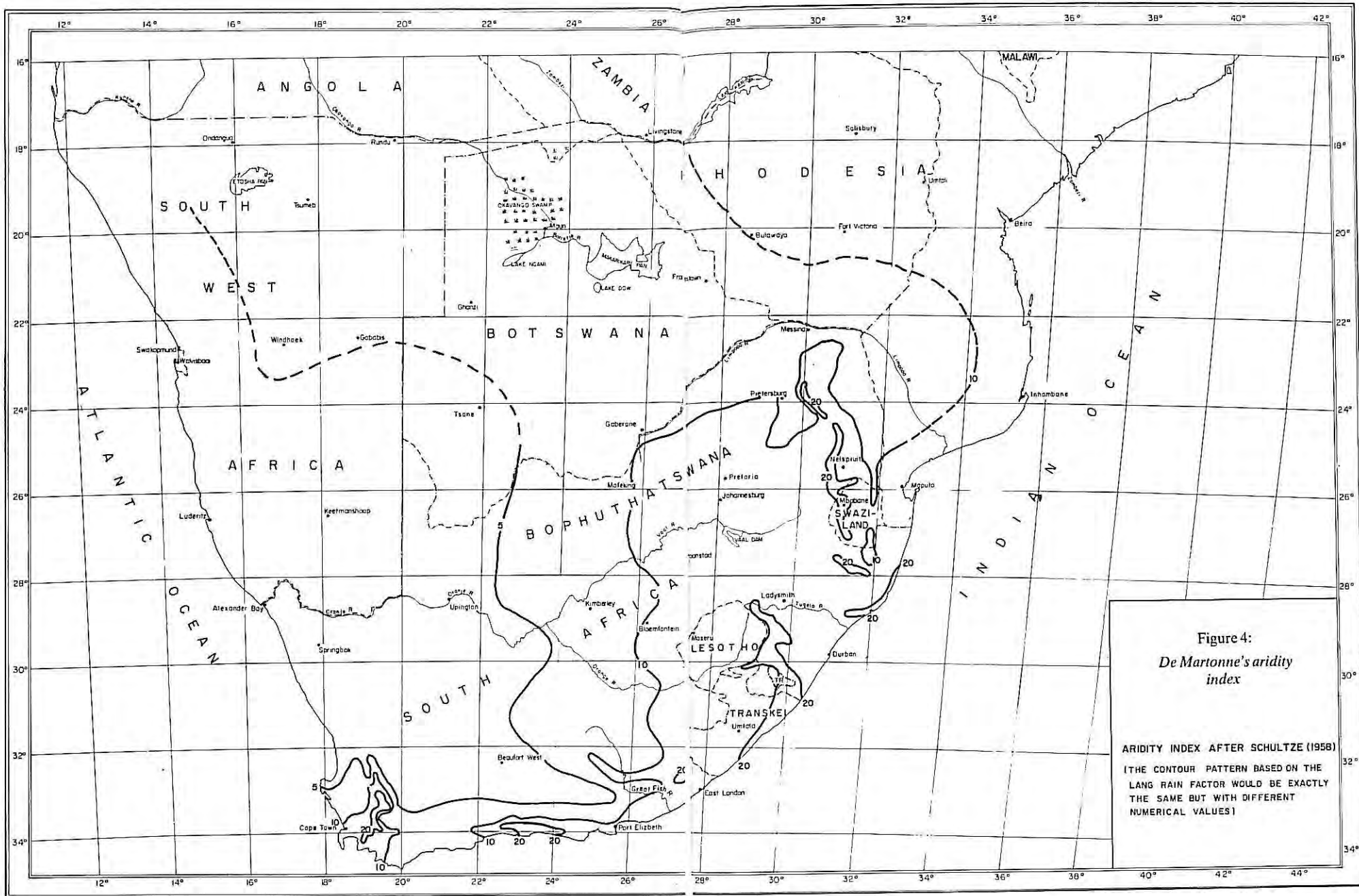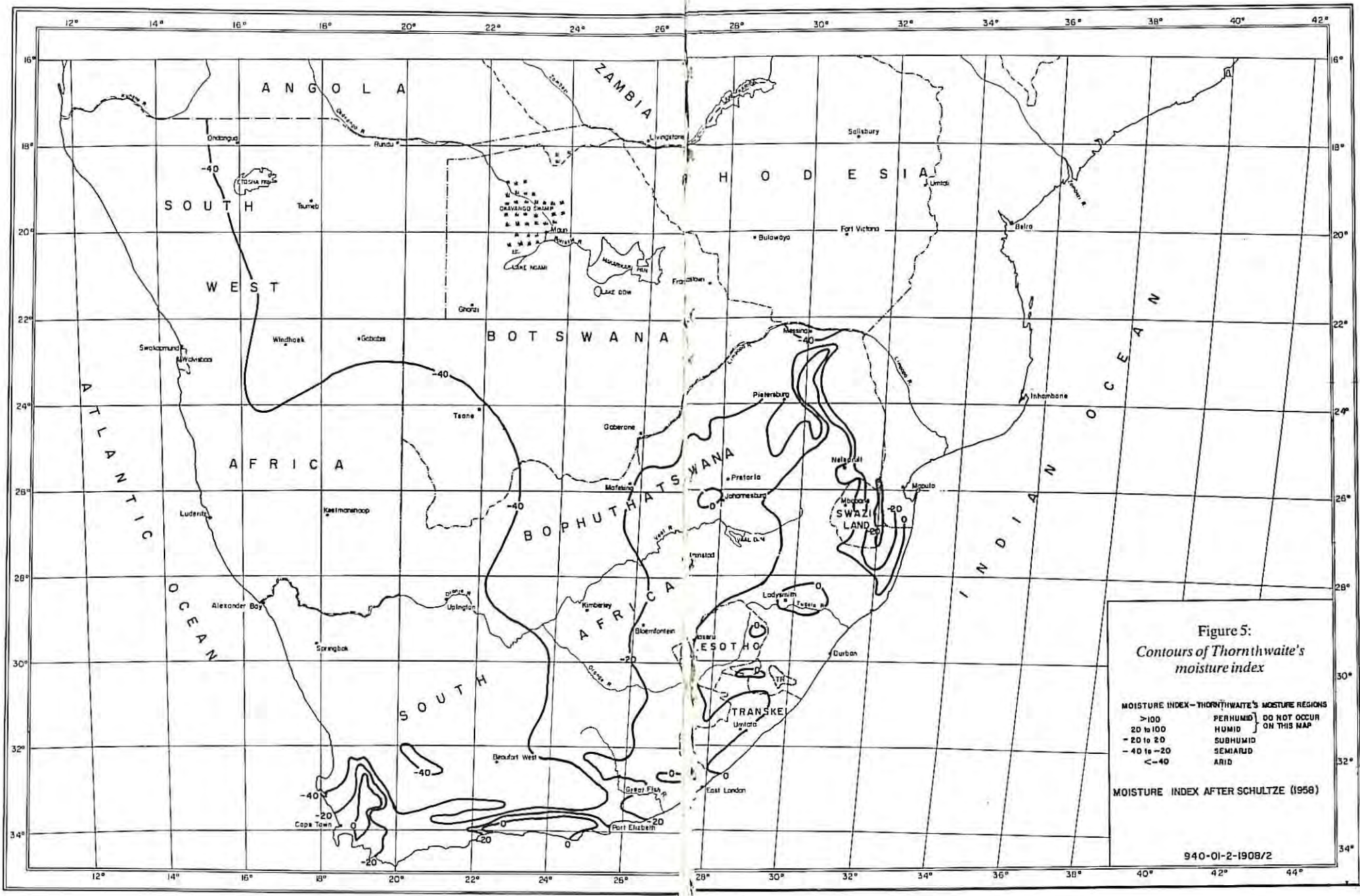
Figure 4:
De Martonne's aridity index

ARIDITY INDEX AFTER SCHULTZE (1958)
(THE CONTOUR PATTERN BASED ON THE
LANG RAIN FACTOR WOULD BE EXACTLY
THE SAME BUT WITH DIFFERENT
NUMERICAL VALUES)

Figure 5:
Contours of Thornthwaite's moisture index

MOISTURE INDEX—THORNTHWAITE'S MOISTURE REGIONS		
>100	PERHUMID	DO NOT OCCUR
20 to 100	HUMID	ON THIS MAP
−20 to 20	SUBHUMID	
−40 to −20	SEMIARID	
<−40	ARID	

MOISTURE INDEX AFTER SCHULTZE (1958)

940-01-2-1908/2

some, although not all, of these roads within a few years of service (**Clauss**, 1967) and there were even cases where the road lasted only a few months. When such failed roads were opened and the base material was compared with the material in the borrow pit from which it originated, a deterioration in the quality and conditions of the weathered dolerite could be observed.

This boundary has always been linked with climate and the original idea was that it could be defined by the annual rainfall. However, the total annual rainfall, which was thought to mark this boundary, varied considerably, being a little above 400 mm in the south, about 500 mm in the middle and 600 mm to the north of the line. It was obvious, therefore, that some modification of the total annual rainfall was required to define this boundary line.

Existing climatic indices, e.g. the Lang rain factor (**Van der Merwe**, 1962), De Martonne's aridity index (Figure 4) or Thornthwaite's moisture index (Figure 5) which had already been determined for South Africa (**Schulze**, 1958), were not satisfactory. Their greatest disadvantage was that they all placed the Eastern Transvaal in the same climatic category as virtually the whole of Botswana and large parts of the Karoo, while large parts of central Rhodesia, including Salisbury, fell into the same category as the Eastern Highlands. This was why a search was initiated to find a new climatic expression which would satisfactorily define the boundary lines between the sound and unsound performance of weathered basic igneous rocks.

The basis of this investigation was the realization that the total annual rainfall alone was insufficient to define this boundary, but that the seasonal variation of rainfall, i.e. rain falling during the warm or cold season or throughout the year, in combination with the potential evaporation, had a great influence on the availability of water as a weathering agent.

The work was concentrated initially in the Orange Free State because dolerite in all stages of weathering was being used for road construction throughout the Province. There were performance problems in the east and none in the west, but the whole Province falls into the summer rainfall area. The work was based on the general observation that, in the west of the Province, the final weathering product derived from dolerite was a gravelly soil with a relatively low fines content composed often of the more or less oxidized mineral constituents of the parent rock, while in the east the product was a clayey soil. These two conditions were separated by a transition zone, mostly 20 to 30 km wide, which crossed the Province in a north-south direction between 26°E and 27°E. A check of the liquid limits and the plasticity indices recorded by the Provincial Roads Department showed further that, in the east, the liquid limit of the soil residual from dolerite was generally greater than 30, and that the plasticity index was greater than 10 and in certain areas even greater than 20, in contrast to the prevailingly lower values in the west (**Weinert**, 1959, 1961). These conditions are shown in Figure 6.

All climatic factors were investigated independently, including those which were unlikely to have an effect on weathering. Separate investigations were done for January, the warmest month, and July, the coldest. Contour maps were drawn for each climatic factor and the patterns of these contour maps were compared with the established boundary between the satisfactory and unsatisfactory road per-
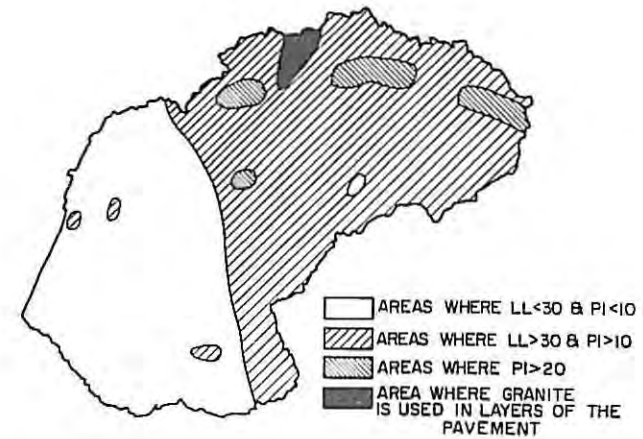

AREAS WHERE LL<30 & PI<10
AREAS WHERE LL>30 & PI>10
AREAS WHERE PI>20
AREA WHERE GRANITE IS USED IN LAYERS OF THE PAVEMENT

Figure 6: *Predominant soil constants of soil residual from dolerite in the Orange Free State*

formance of weathered Karoo dolerite. From this comparison it was obvious that of contour maps for January, only those which referred to moisture or water had a pattern which agreed with the established boundary.

In the meantime, it had become known that all other basic igneous rocks and their metamorphic derivatives performed very similarly in roads to Karoo dolerite, and that the work could safely be extended to include the other basic rocks as well. It was also recognized that the differences between the performances of weathered rocks in pavements could be correlated with the prevailing mode of weathering, i.e. decomposition where the performance was problematic and disintegration where few problems were experienced. The boundary between the sound and unsound performance of weathered basic rocks in pavements was therefore also the boundary between the two modes of weathering: disintegration and decomposition.

The climate investigation was therefore extended to cover the areas to the north and south of the established weathering and performance boundary. Eventually all suitable contour maps were superimposed and critical values were determined for each climatic factor. Only contour maps for January which referred to moisture or water were included in this extension of the study. From an analysis of these maps, the following relation emerged:

$$R = \frac{E_J}{P_J} \quad \text{...................................... (1)}$$

where E_J is the computed evaporation from a shallow freewater surface during January, P_J the total rainfall for January and R stands for 'ratio'.

A contour map of R-values was then compiled and there appeared to be a satisfactory agreement between R = 3 and the weathering and performance boundary.

Up to this stage, the work had concentrated on summer rainfall areas and it was here that the value R = 3 satisfied the conditions. When the study was extended to cover the whole Republic of South Africa, however, it was found that the values for R in the winter rainfall areas were too high in comparison with the predominant type of weathering. Crystalline rocks, i.e. acid and basic igneous rocks and their metamorphic derivatives, decompose in the Cape Peninsula but values of approximately R=9 were obtained for Cape Town. Such high values were to be expected, however, if it is realized that the ratio R refers only to January which, although it is the warmest month, is dry in winter rainfall areas with consequent high values for E_J and low values for P_J.

Use of July, which is one of the most humid months in these areas, instead of January, could have been considered. The R-values in the winter rainfall areas would have been lower, they may have been too low for the type of weathering, e.g. the values would have been less than 1 for R in the Cape Peninsula, because July is also the coldest month and evaporation is therefore low while precipitation is high. The rain-water during this season, however, is cold and its effectiveness as a chemical agent is probably less than that of rain-water received during the warm season, although a higher content of carbonic acid may compensate partly for this. Similar values for R, some determined for the warmest month and some for the coldest, would therefore not necessarily have the same bearing on the mode, rate and intensity of weathering even if the parent rock were the same.

The reactivity of water increases with increasing temperature (**Glasstone**, 1956). The effect of ice on rocks is only physical or mechanical and the colder the water the less chemically reactive it is (**Ugolini** and **Anderson**, 1973). Within the ranges of water temperatures encountered in most parts of Southern Africa, however, a rise in temperature of 10 °C approximately doubles the speed of all chemical reactions. This is part of the reason why the R-values for January agreed satisfactorily with the performance and weathering boundary in the summer rainfall areas. Thus, while retaining January, i.e. the warmest month for Southern Africa, as the principal part of the year for this study, a correction of the R-value for the annual distribution of rainfall had to be considered.

This correction was found in the expression

$$D = \frac{12 P_J}{P_a} \quad \dotfill (2)$$

where P_a is the total annual precipitation. Values of D greater than 1 denote summer rainfall, those of D less than 1 winter rainfall, and when D is near 1, the rainfall is distributed more or less evenly throughout the year.

The multiplication of expressions (1) and (2) then produces the equation which has become known as the N-value (**Weinert**, 1964; 1974):

$$N = \frac{12 E_J}{P_a} \quad \dotfill (3)$$

The particular feature of this formula is the use of two different parameters of time, a month for evaporation and the whole year for precipitation. Basically the formula states that twelve times the computed evaporation of the warmest month, mostly January in Southern Africa, divided by the total annual precipitation gives a numerical value, hence the name "N-value", which can be used to describe the different climatic environments in which rock weathering varies and in which rocks perform differently in roads.

The factor 12 in the N-formula is now only a constant which could really be omitted. Contour maps of N-values (Figure 7) would have exactly the same pattern with or without this factor 12. The factor has been retained, however, in order to keep the N-values so obtained as integers and not decimal fractions.

EVAPORATION

The calculation of evaporation, E, in the above expressions, requires some special comment. There is a variety of formulae for E, many of them involving observations of the dynamics of higher atmospheric layers, or other meteorological factors which are mostly observed only at major weather stations. Furthermore, calculating the evaporation of a station by using different formulae usually produces different results, leaving the question of which of these results is the 'correct' one. Measuring evaporation from pans again gives different results depending on the position of the pan – above, on or sunk into the ground.

After it had been realized that evaporation would have to be included in the solution to the problem, whatever that might be, the question arose: which value for E should be accepted as the 'correct' one? In an investigation like this, which aimed at a numerical expression for a climatic equilibrium, it was felt that the absolutely correct value of E would be of less relevance than the correct relation between the different stations, i.e. whether E at one station is twice or only half as great as at others. A formula was therefore selected, based on meteorological observations which were recorded at an acceptably large number of weather stations in the Republic of South Africa. The formula was then used for all calculations of E. The formula used initially was the one developed by **Meyer** in 1915:

$$E = c(e_s - e_a)(1 + \frac{\overline{Vw}}{10}) \quad \dotfill (4)$$

where E is the monthly evaporation in inches; c is a constant, being 15 for shallow free-water surfaces; e_s and e_a are respectively the saturation and the atmospheric vapour pressures in inches of mercury, which depend on the average air temperatures for e_s and the relative humidity for e_a; and $\overline{Vw}$ is the average wind speed of the month concerned in miles per hour. The determination of the vapour pressures

makes working with this formula rather cumbersome.

A problem when using the Meyer formula is the use of wind speed which is not often observed regularly. Estimates of the wind speed at a number of stations were required. Fortunately, the effect of wind speed in the Meyer formula is not particularly strong and a variation of ± 1 m/s (± 2 miles per hour) is not significant.

Early in the sixties, **Olivier** (1961) published his latitude-dependent expression for daily evaporation which was used in the following form:

$$M_p = \frac{c}{W\emptyset S} \quad\text{...................}\quad (5)$$

where M is the free-water surface evaporation in mm/day from a standard tank, i.e. a shallow free-water surface; c is the average depression of the wet bulb in °C for a particular month; and $W\emptyset S$ is a correction for latitude, called the Monthly Field Water Requirement Characteristic Constant, which is obtained from tables. The Southern African values for $W\emptyset S$ during January are shown in Table 1.

TABLE 1

Monthly field water requirements characteristic constants ($W\emptyset S$) for Southern Africa in January

Lat. S	10°	15°	20°	25°	30°	35°	40°
$W\emptyset S$ January	0,70	0,74	0,76	0,79	0,82	0,83	0,85

The values for evaporation obtained from formula (5) agree only partly with those obtained from the Meyer formula (4), and it was found that the effect of wind had to be considered as well. A wind correction has therefore been developed which reads

$$\text{Wind correction} = 0,58 + kW \quad\text{................}\quad (6)$$

where k is a constant which is 0,14 if the wind speed W is measured in m/s, 0,0626 if it is miles per hour or 0,0719 if it is knots.

This wind correction is 1 if the wind speed is 3 m/s, and for the vast majority of Southern African stations it is between 0,8 and 1,2.

The amended formula of Olivier is then

$$M_p = \frac{c(0,58 + kW)}{W\emptyset S} \quad\text{.............}\quad (7)$$

This formula gives daily values for evaporation and, to cover a whole month, it must be multiplied by the number of days, a, of the month concerned:

$$M_p = \frac{ac(0,58 + kW)}{W\emptyset S} \quad\text{.............}\quad (8)$$

Since most determinations in Southern Africa will be done for January, the warmest month, a is mostly 31, and by replacing E_J in formula (3) with the expression for M_p in formula (8), the following N-formula is obtained (**Weinert**, 1974):

$$N = \frac{372\,c_J\,(0,58 + kW_J)}{P_a\,W\emptyset S_J} \quad\text{...............}\quad (9)$$

where the index J stands for January; in the northern hemisphere this would be July and the same corrections for latitude would apply.

Selection of month for the calculation of E

It has been said that E must be calculated for the warmest month of the year and that this is mostly January in the southern hemisphere and July in the northern hemisphere. This applies to the whole of Southern Africa south of 15°S and it also applies to all other temperate or warm climates with an annual mean temperature of more than 10 °C (**Weinert**, 1965; 1974) in latitudes north and south of 15°.

Where there are two annual zenithal positions of the sun which are separated by two months or more but by less than five months, i.e. between 15° and about 5° latitude on either side of the equator, E should again be calculated for January (or July) which months mostly fall within the major rainy season, even if they are often not the warmest months of the year. January (or July) should also be used for the calculation of E in those few areas between these latitudes which receive winter rain.

Conditions in the zone next to the equator, i.e. between 5°N and 5°S, where seasons are almost non-existent, have not yet been investigated but it is suggested that E be calculated for the rainiest month.

Where the annual mean temperature is less than 10 °C, irrespective of the latitude, E should be calculated for January (or July). The significant values of N, however, may be different from those of the temperate or tropical climates and a correction may have to be considered to retain the numerical significance of the N-value. These aspects still have to be investigated.

Where the annual mean temperature is below freezing point and where rocks do not decompose noticeably (**Ugolini** and **Anderson**, 1973), the N-value loses its significance for rock weathering.

The data required for the calculation of the N-value of a station are:
a) the annual average rainfall,
b) the average air temperature for the month concerned,
c) either the average wet-bulb reading (dry-bulb readings being equivalent to the mean monthly air temperature) or the average relative humidity of the month concerned because either value can be used for the determination of c, and
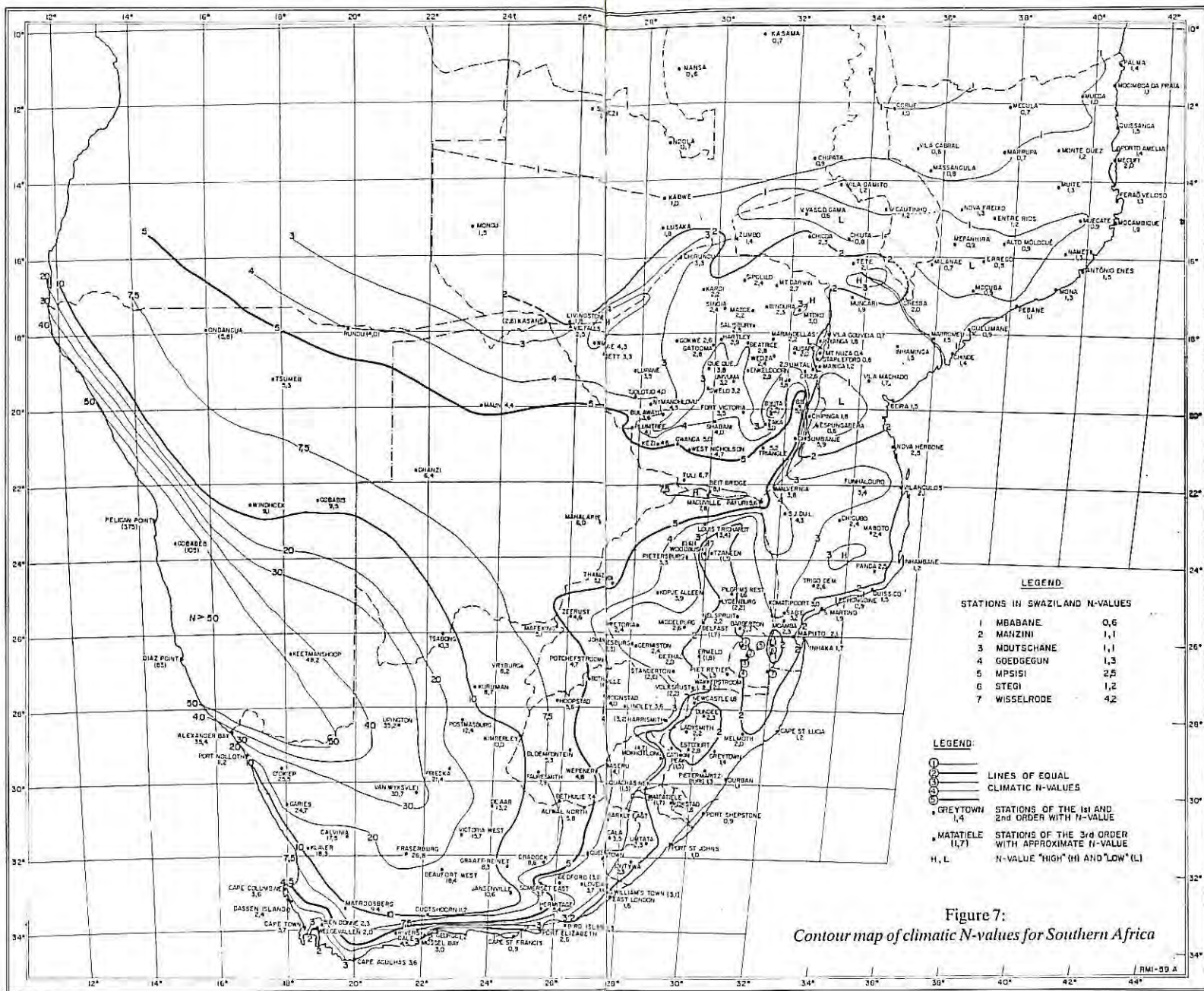d) the mean wind speed of the month concerned.

Figure 7:
Contour map of climatic N-values for Southern Africa

Only integers of N are significant and, amongst the integers shown on the N-value contour map of Southern Africa (Figure 7), only N = 1, N = 2, N = 5 and N=10 are of importance for road engineering (Figure 8). In Rhodesia, N=4 is also of significance while there are no areas where N is more than 10.

Determination of the N-value of a site

A case may arise where the N-value of a site has to be specially determined because of conditions which may result in considerable local differences from the regional N-values as shown on the map (Figure 7). This is particularly to be expected in mountainous regions where the topography may greatly affect both the monthly and annual precipitations, and if this is the case, abrupt changes in the N-values can be expected (Figure 9).

The following procedure is, therefore, suggested for the determination of a local N-value under particular conditions. An automatically recording thermograph, hygrograph and anemometer must be set up in a convenient place. The thermo-
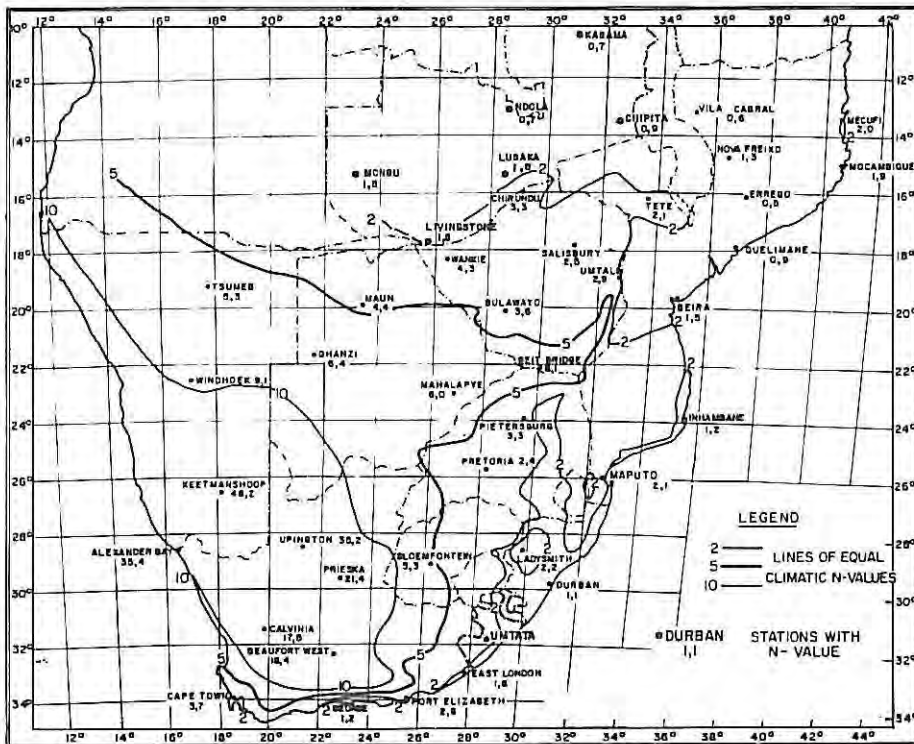
Figure 8: *Contours of the values N=2, N=5 and N=10*

Figure 9: *Areas which warrant local determination of N-values (hatched)*

graph and the hygrograph must be kept in a Stevenson screen, while the anemometer must stand free so that it can be fully affected by the surface winds. The instruments require little attention during the period of observation; the recording charts must be changed once a week and the hygrometer hair washed at the same time.

The observation period need only be a full month of January during the time of preparatory investigations for the road. Some corrections to the observed results may become necessary in the case of an unusually humid or dry January. Information on the long-period norms for the temperature and relative humidity can be easily obtained from nearby weather stations, because there is a good network of such stations over the whole country. A comparison of the long-period averages of such a station with the deviation for the particular January under observation, will provide a means for a pro rata correction of the temperature and humidity data obtained from the new site. A similar correction may have to be applied for the wind speed. Although many weather stations in the country are now equipped with anemometers, a reliable long-term mean for the wind speed may not yet be available. Such a mean value should be based on at least five, or better still, ten or more years of observation. Care should therefore be exercised with the determination of the mean wind speed. When the observation periods are short, wind

speeds from a number of pre-existing weather stations around the new one may have to be employed so that their deviations for the January concerned can be averaged out with a view to obtaining a wind speed correction factor for the new station. It should be remembered that the wind speed is not very sensitive in the N-formula (9) and that an error of $\pm$ 1 m/s has little effect. The annual precipitation is read from maps prepared by the relevant Weather Bureau.

GEOLOGY

The major subject under discussion, that is natural road building materials, are of course 'geological environment' in themselves but it must be realized that minerals, rocks and soils are only a part of this geological environment. Although rocks and soils are of prime importance to the road builder, and although they are as much in danger of being exhausted by man as are many other natural resources, other geological features must also be considered when planning, designing and constructing a road.

The important geological features, besides rocks and soils, which influence a road in many ways are bound to be the structural conditions of the crust. The strike and dip of strata, disconformities, unconformities, discordances and further features derived from tectonic movements, such as folds and faults, all influence the structural design and the performance of a road by affecting the stability of slopes, the foundations of bridges, seepage (*Plate 7*) and the situation of suitable construction materials.

The local geology thus largely determines the choice of construction materials and the details of the structural design. In both cases there is interaction between various natural conditions, especially geology and climate; the mode and rate of weathering and the type of residual soil depend on the parent rock and the way in which it is affected by the prevailing climatic conditions.

The local geology can, of course, be read off maps; it should be realized, however, that the gological map of South Africa, scale 1:1 000 000, usually shows only the geological conditions in their age relationship, while the road engineer is much more interested in the local lithology, i.e. the types of rock and possibly soil which are available. To the road engineer, it is of lesser relevance to know that this route crosses Archean or Cape granite (although their engineering properties differ a bit because of mineralogical, textural and structural differences) than to know that granite is available and how granite weathers and behaves in different climatic and topographical environments. Even less information on the materials actually available can be gained directly if the map shows nothing but Ecca, Beaufort, etc. which indicates only that, whatever the country rock is, it has been deposited or formed during a certain time span. In such cases, unless the user is familiar with the lithology of a particular geological formation, the legend must be studied carefully and, if available, reference should be made to the explanations of the map.

Geological maps of a larger scale, in the Republic of South Africa nowadays mostly on the scale of 1:250 000, occasionally 1:50 000, and formerly 1:125 000, of-

ten provide better information on the lithology directly, though grouped according to the geological age, but reference to the explanations of the map is again recommended.

Soil engineering maps (**National Institute for Transport and Road Research,** 1978a) would, of course, provide the best and most detailed information. They are prepared, however, only in association with specific road projects and it will take a long time until so much of the country has been covered that a new route will cross country which has already been surveyed in such detail as shown on the soil engineering maps. The information supplied on such maps may, of course, be applicable to other sites and areas as well. Materials data obtained from soil engineering maps and other sources are therefore stored in data banks. The methods of storage and retrieval are not the same everywhere (**Holden,** 1971; **Clauss** and **Vail,** 1975); however they are based on the observation that the mode of weathering of a particular rock in a given climate, expressed by the N-value, and under given topographical conditions, described in simple terms, is predictable, as is the rock's performance in a road structure. Storing materials data according to lithology, climate and topography then, provides a means of supplying information about materials which occur in similar, but so far unsurveyed areas.

No truly lithological maps are prepared in Southern Africa although they would actually be what the engineer requires. Stratigraphy is the principal basis of the mapping units of all Southern African geological maps, although the larger-scale editions also show a fair amount of detail about the lithology; soils or other unconsolidated materials, however, are shown only where their cover is very thick. The mapping units of soil engineering maps are based on lithostratigraphy, distinguishing for instance between Ecca shales and sandstones, and Beaufort sandstones, shales or mudstones. They also differ from the usual geological maps by showing, in addition, the surface cover of unconsolidated materials such as sand, clay, colluvium, etc. The details of their production have been described in TRH 2 (1978): 'The production of geotechnical and soil engineering maps and the storage of materials data' (**National Institute for Transport and Road Research,** 1978a). Topography, particularly the gradient of slopes which is important in regard to (a) the possible thickness of weathered material, (b) the stability of such slopes, (c) drainage and (d) the most economical location of borrow pits and quarries, may be added to such a map or shown on a separate overlay.

'Geotechnical map' is the not-entirely-satisfactory name adopted in South Africa for maps which cover those geotechnical aspects that are required for route location; their production has also been described in TRH 2 (1978) (**National Institute for Transport and Road Research,** 1978a). These maps usually cover a much larger area and a much greater length of proposed road, up to several hundred kilometres. They are produced, therefore, on a much smaller scale than soil engineering maps and they show only as much detail as is required to locate the route in the geotechnically most economical position. These maps normally form part of a much wider survey which includes items such as ecology, nature preservation, social and political aspects, and many more.

Engineering geological maps for township planning differ from those for road planning and design particularly in that sources of natural construction materials

are of less, and the occurrence of suitable and unsuitable foundation conditions of greater, importance.

The various types of geological map thus represent a summary of what should be understood as the 'geological environment', and if they and their legends can be read properly, they can provide much information which would otherwise have to be collected with difficulty in the field. The results which are obtained from the interaction between the geological conditions and the climate, together with crustal movements, are then reflected in the various surface forms.

TOPOGRAPHY AND DRAINAGE

Topography and drainage are closely related and they are both dependent on climate and geology. It is obvious that topographical forms are primarily the result of weathering and of the difference in the resistance of rocks to the weathering agents. Flowing water also selects weaker rocks or weaker zones, e.g. faults, in which to erode its drainage channels. In general, therefore, outstanding ridges indicate the strongest rock in the area while the thickest soil-cover can be expected in the depressions. Besides on steep slopes, access to the solid country rock can often be gained near or on the floor of drainage channels, where erosion is most effective, and in countryside with little relief this is often the only place to inspect these rocks in situ.

The general drainage pattern, as revealed on maps or air photos, indicates the overall gradient or gradients of the area. On maps or air photos where neither the source nor the mouth of rivers and streams, perennial or intermittent, is shown, the confluence of two rivers or streams points in the direction of the flow and gradient in the vast majority of cases. Since exceptions may occur under special conditions, it is necessary to compare a number of such confluences to avoid misinterpretation through an occasional deviation from this rule.

The slopes of hills and their gradient attain a particular importance in the search for, and the chance of obtaining, suitable pavement materials especially if the climatic environment has an N-value of less than 5 and the materials have to be derived from rocks which are decomposing, i.e. mostly acid and basic crystalline rocks. The faster run-off, superficial as well as internal, on a sloping surface affects the weathering residue on rocks in several ways.

Fines are lost and carried away, leaving only heavier particles behind. If the rock concerned contains minerals which do not decompose, or decompose very slowly, e.g. granite, the resulting washed-out soil is liable to collapse, erode or slide if its delicate equilibrium is disturbed.

Montmorillonite is the final product of decomposition from basic crystalline rocks in waterlogged areas where N is between 2 and 5. If the drainage improves, however, as is the case on slopes, the montmorillonite in the top layer of the residual soil changes into kaolinite although montmorillonite still exists lower down.

The possible effect of varying gradients can be explained from two boundary conditions. Where there is no gradient, i.e. where the country is absolutely flat, waterlogging occurs especially where N is less than 5 and weathering is deep.

When the gradient is vertical, i.e. 90°, only bare rock is exposed and an oxidation film is the only sign of weathering which may be present; all loose pieces of rock will have fallen off the face. Between these extremes, the thickness of the soil cover, as well as that of the various layers of weathered material, should decrease from a maximum value in the flats to nil on a steep slope. These are, of course, general statements since the shape of the slope also plays a role. On a concave slope, for instance, a certain low-angle gradient will be found near the pediment where the depth of weathering will be in accordance with the climatic environment, and where the cover of debris may be thick. On a convex slope, on the other hand, a similar low-angle gradient will occur on top of this slope with increasing steepness and run-off downhill; consequently this particular low-angle part of the slope will be weathered only slightly or not at all and very little if any debris will be found.

These aspects were investigated in some detail on hill slopes underlain by norite in the vicinity of Pretoria where N=2,4 and by granite in the Eastern Transvaal Lowveld where N is less than 2. This investigation permitted an approximate definition of a few characteristic gradients of slopes. It appears more feasible to use degrees, and not percentages, for the classification of natural slopes which might be very steep, thus avoiding very high figures.

Only flat country where the gradient is less than 5° can rightly be considered as waterlogged. Weathering is deep in accordance with the climatic environment and montmorillonite is the principal clay mineral formed on decomposing basic crystalline rocks; it is the final decomposition product where N is between 2 and 5.

Where the gradient of the slope exceeds 5°, the montmorillonite in the surface layer of the residual soil on basic crystalline rocks changes into kaolinite. This indicates that run-off has taken effect. The thickness of the kaolinite is intially only about 50 to 100 mm.

Where the slope exceeds 15°, the thickness of the kaolinite increases markedly, indicating a further noticeable increase in the run-off and internal drainage with a greater effect downward. The thickness of the kaolinite layer may now increase to more than 500 mm.

When the slope is more than about 30°, very little soil is retained on the surface and the thickness of weathered rock is also slight. With further steepening of the slope, increasingly more fresh rock appears on the surface and from about 45° onwards weathered rock and soil virtually disappear. When the slope is more than 60°, there will always be outcrops of fresh rock.

These findings are of practical importance when considering the precise location of borrow pits and quarries. Provided the thickness of a particular rock is sufficiently great, suitable material for each purpose of road construction can be obtained almost everywhere. This is so because whatever the final product of weathering is, at some depth unweathered rock must be encountered and there is a gradual transition from one terminal condition to the other. The recovery of suitable material is therefore a geological problem only when either the condition of the fresh rock itself is borderline or even unsuitable, or the whole of the rock layer is thoroughly weathered from top to bottom as is occasionally the case with dolerite sills in areas where N is less than 5.

The classification of slopes in terms of their steepness has already been undertaken by a number of workers, e.g. **McHarg** (1969), **Nefedova** (1973) and **Brunsden** *et al* (1975), and there are noticeable agreements between them although they worked independently. It appears therefore that a practical grouping of the gradients of slopes can be set up. Such gradients should also be shown on maps.

0° to 5°   This is country where waterlogging soil conditions may prevail, particularly if the soil is clayey. Weathering attains its greatest depth within the prevailing climatic conditions and the soil cover, transported or residual, is thickest. Borrow pits will have to be holes in the ground with difficult access for trucks and the pits are likely to be flooded in rainy weather.

This group may be subdivided further with the view to geometric design as well as other planning purposes, such as industrial developments. A practical subdivision would be 0° to 2° and 2° to 5°.

In this subdivision, 0° to 2° would be terrain which could be regarded as entirely flat with little natural run-off and which will have the associated problems for surface drainage. This range also approximately accords with the permissible gradient of terrain suitable for the planning of large industrial areas which have to be supplied with their own railway links. It may be mentioned in addition that in agriculture, contour walls are required when the slope is more than 3°.

The range from 2° to 5° still does not cause real problems in road alignment and the surface drainage improves, although waterlogging still occurs.

5° to 15°   Surface drainage is satisfactory, and there is no waterlogging. The soil cover is thinner and better-quality material is closer to the surface. The soil may, however, be mixed with boulders. This range has the best conditions for borrow pits, providing for relatively level access and satisfactory drainage.

Road construction will often require cuts and fills unless the centre line can be located close to the contours. 15° is also the upper limit for the development of residential areas.

15° to 30°   The soil cover is thin, consisting mostly of good-quality material but there are frequent protrusions of hard fresh rock. Surface drainage is fast and erosion pronounced. This range gives the best site for quarries because the fresh rock is easily reached and there is little overburden, while access can still be provided for trucks and machinery; it provides difficult sites for borrow pits.

Such a terrain naturally poses great difficulties for the planning and construction of roads and many deep cuts and high fills will be required.

> 30°   Such slopes consist mostly of rock outcrops, and no soil is retained, or only very little soil in places. These slopes provide ideal quarry sites if the access can be kept on less sloping ground (i.e. the quarry floor should be situated at the base of the slope) and provided undercutting does not cause collapse.

The road engineer will, of course, avoid this terrain wherever possible. Should the road have to cross such slopes, however, serpentines and tunnels will be required.

FLORA, FAUNA AND MAN

The effect of plants, animals and man on the quality of natural road building materials is not great – except perhaps for the fact that man removes the suitable materials – but a few points deserve mentioning. These points refer mostly to the natural subgrade which is only a natural road building material inasmuch as it undergoes some preparation prior to the actual construction and has a certain influence on the performance of the road.

VEGETATION

The disintegration of rock is aggravated by the presence of plants whose roots grow into cracks and fissures, and assist in opening them up. This effect occurs everywhere that plants can exist which means that it is practically absent only in desert regions, i.e. where N is more than 10.

A dense cover of vegetation, as occurs where N is less than 5, supports decomposition by the addition of decaying plant material, often in the form of humic acid, and by bacterial action.

The climate-dependent differences of the vegetation cover have a marked influence on the ease with which road building material can be located. Particularly where N is less than 2, as is the case in most of Natal, the Eastern Transvaal, the Eastern Cape Province, Transkei and the eastern parts of Rhodesia, the density of vegetation makes it difficult to locate suitable construction materials on air photos and in the field. Even where N is between 2 and 5, much may still be camouflaged by vegetation while, where N is greater than 5, rocks and soils are well exposed and they can be traced with much less difficulty on air photos and in the field.

Vegetation may have a severe influence on the physical properties of soils and thus on the natural subgrade of a road. These influences are mainly associated with the effect which plants have on the moisture regime of the soil.

Supposedly, soil creep is aggravated and slope stability reduced by a dense cover of trees (**Brown** and **Sheu**, 1975) whose total mass increases the creep rate and, together with wind load, decreases slope stability. An immediate effect of deforestation then is the decrease of the creep rate and the improvement of slope stability. With the decay of the root system and the rise of the water-table due to the drop of evapotranspiration, however, the creep rate will increase and the slope

stability decrease again some time after deforestation, unless a ground-cover can be re-established quickly, thus providing a new anchorage and depressing the ground-water level (**Harmse** and **Van Zyl**, 1975).

Plants cause a great loss of soil moisture by evapotranspiration. A few figures may serve as an example (**Henrici**, 1945/6, 1947; **Williams** and **Simons**, 1963): the amount of water lost during a sunny day by evaportranspiration from

> a gum tree (Eucalypthus macarthuri) is 450 to 500 ℓ/day
> a black wattle (Acacia mollissima) is about 250 ℓ/day
> deciduous fruit trees is 5 ℓ/m²/day
> highveld grass (Themeda) is 7 ℓ/m²/day

Such a loss of moisture leads, of course, to the desiccation of the soil. When the vegetation, particularly the trees, is removed, as happens when a site is cleared for construction work, the moisture content of the soil increases. This increase will result in heave if the soil is an expansive clay and it may lead to collapse settlement in the case of a number of transported and residual soils. The removal of vegetation should therefore be done well in advance of the beginning of the construction, possibly allowing the fallow soil to be exposed to at least one or perhaps two rainy seasons (**De Bruyn**, 1973, 1975).

The desiccation caused by trees (especially gum trees and wattles) which grow after the construction of, for instance, a road, may make this road crack and it may be observed that the most severe crack tends to occur roughly below the perimeter of the crown of the tree: this line coincides roughly with the extension of the root system.

ANIMALS

Animals may also aggravate rock disintegration. Digging animals, ants and termites being the most effective, remove large quantities of fine material from the underlying strata and carry it to the surface. This material then forms the biotic surface soil which is so common in Southern Africa. The influence of animals on decomposition is only of an indirect nature: in carrying fine material to the surface for some time, where its consistency is loose to very loose, they increase internal drainage and infiltration of water.

The stone lines which almost always develop from such biotic action and which have been described in Chapter 2, become aquifers in which water is stored as long as they remain confined within the soil. When opened, e.g. by a road cutting, they naturally change into zones of seepage.

Larger digging animals like moles (*Plate 8*) and others may cause damage to structures like roads or airfields. Although more conspicuous, the effect of the work of such larger animals is of less importance than that of the innumerable insects and other small animals.

MAN

Man's influence on environmental conditions is strong and it may even be such that a new environment is created. This does not apply only to built-up areas: man's influence on the natural equilibrium through the building of roads, railway lines and canals, by developing irrigation schemes, and through many other activities has a multitude of often unexpected effects.

It has already been said that the removal of vegetation has a profound influence on the stability of slopes, the water-table and the moisture regime in soils. Similarly, road cuttings, if not designed carefully with regard to the type of rock, the dip of strata and zones of weakness such as faults, always disturb a natural equilibrium and a noticeable decrease in the slope stability may result.

Karoo mudstones, as long as they are not disturbed, are in a state of equilibrium with their environment. When exposed to air, however, e.g. in a road cutting, they disintegrate quickly and break into small, more or less cubical pieces which tend to fall down if the slope of such a cutting is too steep. The mechanism which causes this particular form of disintegration is not yet fully understood and several explanations have been offered; they will be discussed in Chapter 8. This disintegration comes to an end when the cover of crumbled rock is such that the original equilibrium, particularly that of moisture and overburden pressure, is reestalished.

It should also not be overlooked that man's water schemes, such as irrigation and large dams, have an influence on the local climate. Such schemes may make the local conditions turn more humid than before; this will encourage vegetation and lead to an increase in evapotranspiration with consequent effects on the local circulation and the water-table. In 1959, during the investigation of a particular crack pattern in a road south of Kimberley, i.e. in an environment of N = about 11, it was found that the irrigation at the adjacent agricultural research station caused a particular, local pattern of seasonal expansion and shrinkage in weathered Dwyka boulder mudstones. In addition, the impression was gained that the previously disintegrating dolerite in the vicinity showed signs of decomposition. Such a change from predominant disintegration to decomposition would, of course, be a long-term effect and, since this station was then only about ten years old, no certainty about this observation could be obtained. The quantity of water used for irrigation was approximately equivalent to what would be expected from normal rainfall in an area of Southern Africa whose N-value is between 3 and 4.

The impact of man on the environment also affects the availability of natural road construction materials. The very existence of people who have developed beyond the cultural state of gatherers and hunters forces them to shape their environment according to their needs. The first step in every 'development', whenever man enters virgin terrain, is the provision of means of communication. Therefore man builds roads and for these roads he requires increasingly more materials. Most of these materials are obtained from nature and it is obvious that the exploitation starts with those rocks and soils which are best suited for the purpose. In many parts of the world, particularly in parts of Europe and North America but also in South Africa as for example in the Witwatersrand area and the Orange

Free State, the most suitable sources of natural road construction materials are almost exhausted, forcing the road engineer to look for other materials or to turn to sources which have previously been rejected.

WEATHERING

GENERAL

'Weathering' is to be understood here as that alteration which occurs in rocks due to the influence of the atmosphere and hydrosphere (**Legget**, 1962). In petrology, however, the alteration which rocks experience due to unloading if they move nearer to the surface, is also called 'weathering'. Because of periodic crustal movements, rock masses may subside at one time, during which they are covered and compressed by an ever-increasing layer of debris, and then rise at another time, during which the rock mass may eventually move above sea-level, while erosion and denudation lead to unloading. During the subsidence and with the increasing load the rock is compressed, its strength as well as the strain in the rock increase, and the whole mass is so brought into equilibrium with its subterranean environment. The rock reacts like an elastic body: unloading relieves the pressure and the compressed rock expands, which causes it to fail by tensional stress along fractures, many of which run parallel to the surface. The closer the rock is brought to the surface, the denser this network of fractures becomes. Rocks which possess some form of foliation, as do many sedimentary or metamorphic rocks, part mostly along the planes of foliation while homogeneous rocks usually break vertical to the plane of unloading, which in most cases is the actual surface of the earth. The result of this process is that each rock on the surface contains some disturbance which is largely physical in nature.

Certain minerals, particularly in igneous rocks, form at great depths as phases which are stable only under high pressure and temperature. This is more true in basic and ultra-basic rocks than in acid rocks and it applies in particular to the mafic minerals. If such rocks are moved upwards, this equilibrium is distrubed and the more the new conditions differ from the original ones, the more the mineral is inclined to adapt itself to the new environment. Thus olivine, that member of the rock-forming minerals which is stable at a high temperature (in the order of 1 800 °C) and pressure, is easily affected by such a change and is often already transformed into the mineral serpentine before the rock has actually been exposed to the atmosphere. Serpentine, as well as any other minerals developing from this type of alteration, requires a larger volume than the original crystal and this mineralogical change therefore contributes to the fracturing of rocks due to unloading.

From the above considerations it follows that there is actually no truly 'fresh' rock on the surface of the earth and the term 'fresh' should therefore be taken as a description of that condition in which a rock starts to be exposed to the atmosphere and hydrosphere. It has already been said that the above conditions are often described as 'weathering'. However, this term will be restricted in the following discussion to only those changes which are caused in rocks by the actual surface conditions, i.e. when the rock is exposed to the atmosphere and hydrosphere (**Legget**, 1962). Under these conditions a rock may either disintegrate or decompose, or both (**Polynov**, 1937; **Reiche**, 1950).

TERMINOLOGY OF WEATHERING

The terminology should be clarfied at this stage. In the description of samples of rocks and soils, as well as natural road building materials, confusion of the terms 'weathered', 'disintegrated' and 'decomposed' should be avoided. These terms are distinctly different and the misuse of them, when describing a material, can lead to serious misinterpretations.

Weathered is the general term which should always be used if one cannot be, or does not wish to be, specific about disintegration or decomposition.

Disintegrated refers only to the physical break-down of a rock. The term, which could apply to any rock as will be explained below, should only be used if the rock is breaking down into ever-smaller pieces while the minerals remain unaltered, although a thin oxidized film on them is permissible.

Decomposed refers only to the chemical weathering of rocks or, more specifically, to the alterations of minerals, and it can occur only in certain rocks in certain environments. This term is therefore much more restricted in its meaning than either of the others but, because of the very great effect of decomposition on the quality of rocks for road building purposes, its correct application is imperative. 'Decomposed' can only be applied to acid and basic crystalline rocks, and in a few exceptional cases to such rocks as tillite, greywacke and certain volcanic effusives, if they weather in an environment of N less than 5. The term cannot and must not be applied to other rocks and, for instance, 'decomposed quartzite' is an impossibility in itself; it could only be 'disintegrated' or 'highly disintegrated', or 'weathered' or 'highly weathered' quartzite.

This does not exclude the possibility that certain sedimentary rocks contain minerals which may decompose, e.g. the feldspar in an arkose. Given suitable environmental conditions, such a mineral will of course decompose, i.e. change into another mineral. However the total quantity of minerals in such a rock is usually low and, moreover, most of them will be of a rather resistant type, e.g. orthoclase, so that the prevailing mode of weathering is still disintegration in relation to the road performance of such rocks.

It has also become customary to use 'weathered' and 'decomposed' as qualifying terms with reference to the progress of weathering: 'weathered' then means the less advanced stage of weathering and 'decomposed' the more advanced one within the same profile. This is not only confusing but wrong and it should be avoided at all costs. 'Disintegrated' and 'decomposed' refer respectively to the physical and chemical mechanisms which act in weathering and they should only be used with reference to these; if there is uncertainty, the non-specific 'weathered' should be applied.

Carbonate rocks, such as limestone or dolomite, and salts dissolve in the presence of water. In the light of the above, the question arises whether the *solution* of these rocks should be considered as a form of decomposition. In the course of de-composition as defined above, a certain degree of solution takes place and, especially in the final stages of the alteration from fresh rock into residual soil, some dissolved components are lost and reprecipitated elsewhere. The major characteristic of decomposition, however, especially as it affects the quality of natural road building materials, is the in situ transformation of primary into secondary minerals, often without changing the texture of the rock; e.g. a feldspar crystal may eventually change into clay although the shape of the original feldspar will still be retained.

The solution of carbonate rocks and salts is thorough, the dissolved material is usually carried away and a 'fresh rock', perhaps cavernous, is left behind. Consequently, there is no formation of new, probably undesirable minerals in the dissolving rock. The dissolved material is reprecipitated, often, although not always, as the same type of mineral as the dissolved one, e.g. a dissolved calcite will be reprecipitated as a calcite, perhaps through an initial stage of aragonite, but a dissolved dolomite will be reprecipitated as one of the modifications of calcium carbonate while the magnesium carbonate component of the original dolomite will remain dissolved. The road engineer is concerned only with the rock which is left behind. If this rock has been weathered in situ, the mode of weathering which affects the road construction properties of the rock is disintegration.

DISINTEGRATION

The mere physical break-down of a rock is called disintegration and it is obvious that this can occur in all types of rock in any environment. Unloading creates a basic system of cracks and fissures along which the rock continues to crumble. There are several mechanisms which make a rock disintegrate and which can act alone or in combination. It should be noted, however, that the forces which cause erosion, i.e. flowing water, wind and glaciers, are not considered in this context. Although rocks may be destroyed by erosion processes, this is not disintegration in the sense described below because the weathering products are removed quickly and no weathering profile or residual material develops.

TEMPERATURE

Diurnal variations of temperature, especially if they are great, have an influence on rock disintegration, though to a lesser extent than was previously thought. Only a relatively thin surface layer of rock is heated during the day while the interior remains fairly cold; the considerable strain developed at the interface of the warm and cold material leads to failures and the development of horizontal cracks which occur in addition to those already caused by unloading. During the night, the process reverses and the outer layer cools faster than the interior of the rock. Consequently, the outer layer shrinks more than the interior and numerous vertical cracks develop. This process leads to 'block' disintegration so called because of the shape of the disintegrated material.

In contrast to earlier opinions, variations of the coefficients of expansion of the different minerals do not now appear to be sufficiently great to have a marked effect on rock disintegration.

GROWING CRYSTALS
Ice

The most generally known process of the break-down of rocks due to growing crystals is the effect of freezing water in cracks and fissures. Again this process is bound to the diurnal variation of temperature which must allow for the alternating thawing and freezing of water. Moisture contained in cracks and fissures is under higher pressure than ordinary surface water and, if the air temperature drops below freezing point, initial freezing occurs only next to the water-air interface, leaving the water further inside the crack or fissure in a closed system. In such a system, water can remain in the liquid phase down to -5 °C and freezing then occurs spontaneously. At this point the volume of the ice increases rapidly by nine per cent of the volume of the water, and the pressure within this system may approach 200 MPa (**Dapples**, 1959).

Salts

Salt weathering occurs particularly in coastal areas and in deserts where cracked or porous rocks are very vulnerable. The number of salts which may assist in the disruption of rocks is large and includes various carbonates, chlorides, nitrates and sulphates of barium, calcium, magnesium, potassium and sodium. Such salts enter the fissures or pores in rocks in solution and crystallize when the water evaporates. Frequent cycles of drying and wetting as found especially in coastal areas and, in Southern Africa, in the eastern margin of deserts, results in the continuous addition of more dissolved salts without removing those already present.

Where salts have developed in confined spaces, such as in the cracks or pores in a rock, they exert pressures which are sufficient to make rocks fail. These pressures can be considerable and it has been observed that the crystallization pressure of calcite, which does not produce excessive pressures in comparison with other salts, is at least equal to the tensile strength of quartz which means that 150MPa may be exceeded (**Rothrock**, 1925; **Goudie**, 1973).

In addition to the pressures caused by the growing crystals of salt, the disruption of rocks may also be caused by the thermal expansion of salts. The volumetric expansion of chlorides and nitrates of sodium and potassium when heated is particularly great. Within the possible diurnal temperature variation in the top surface of rocks, which may approach 100 °C, the increase in volume of these salts can be as much as one per cent, which is considerably more than that attained by rocks (**Cooke** and **Smalley**, 1968).

A third force which is sufficiently great to disrupt rocks is caused by salts which are hydrated and dehydrated from time to time. Sulphates are particularly sensitive to such changes within the diurnal temperature ranges encountered in arid climates. The magnitude of the increase in volume depends on the varying amounts of water which can be absorbed by the crystal lattice. An idea of this magnitude may be obtained from chemical notations such as $MgSO_4.7H_2O$ (epsomite) or $MgAl_2(SO_4)_4.22H_2O$ (pickeringite).

The effects of the latter two mechanisms, thermal expansion and hydration, have not yet been studied in much detail under natural conditions, but it can be assumed with a good degree of certainty that they play a role in rock disintegration. It can also be assumed that the growth of crystals from solution provides the more important mechanism of rock disintegration caused by salts in coastal areas and in the eastern margins of Southern African deserts, while the other two mechanisms may be more important in the arid areas proper (**Cooke** and **Smalley**, 1968), i.e. areas where N is at least more than 10 but probably more than 20.

Secondary minerals

At this stage something which does not fall entirely into the field of disintegration should be mentioned: secondary minerals, e.g. serpentine or clay, which develop during decomposition, also possess a larger volume than the original primary minerals. The forces developed during this transformation are sufficient to disintegrate the weaker parts of the rock.

PLANTS AND ANIMALS

The roots of plants, from grass to trees, can also widen cracks and fissures in rocks. The influence of digging animals is minimal; they remove loose pieces of weathered material and so open up cracks and fissures and support the access of other weathering agents.

RESULTING MATERIALS

All the processes described above only affect a rock physically and this includes some of those where water is also involved. The result is a material which is reduced to an ever-smaller grain size (see *Plate 12*). The final product is a gravel or sand-size material which is composed ideally of the unaltered minerals or mineral aggregates of the original rock, often but not always covered by a thin film of oxidized or hydrated material. Starting from the fresh rock, the spacing of the cracks and fissures narrows and the arrangement of these factures, which was initially along three more-or-less vertical planes, becomes increasingly irregular. The numerous cracks gradually become interconnected and the rock breaks into individual, angular pieces of often approximately cubical shape. The pieces of broken rock continue segregating and, eventually, the break-down may reach the stage where the individual minerals part.

The fragmentation of a rock into its individual minerals need not, of course, necessarily be the end of disintegration. The minerals themselves may continue disintegrating since all or many of them may already have been subject to stress due to a variety of causes prior to the disintegration of the rock. This applies to all minerals including quartz which is generally considered as particularly resistant to

weathering. Although quartz is most resistant to decomposition, it has been shown by a number of authors, amongst whom **Smalley** (1966, 1974), **Moss** (1973) and **Moss, Walker** and **Hutka** (1973), that its resistance to physical break-down is not very great and that this is a possible explanation for the vast masses of silty quartz amongst sediments.

Disintegrated rock in each state of break-down yields road building materials whose quality is closely equivalent to that of the original rock. Oxidation and occasionally hydration or carbonation are the only tokens of some chemical influence which may have occurred at some time but, since they occur only as a film on the surface of the rock pieces or loosened minerals, the overall conditions of the material is that of crushed fresh rock, i.e. nature has done the job of the crushing plant. Rocks whose disintegration has been caused by salts must be treated with care since even small quantities of such salts, retained in the material, may cause severe damage, especially to surfaced roads (see Chapter 10).

DECOMPOSITION

Decomposition is that type of weathering which affects certain minerals chemically and the more such minerals there are contained in a rock, the more radical its overall alteration will be. The most important agent of decomposition is water. Although water is intuitively considered to be a chemically neutral substance, a very small portion of the H_2O-molecules is always dissociated into the ions H^+ and OH^-. Moreover, natural water, either rain or groundwater, is never pure and mostly contains some acid, especially of the carbonic or humic types. Such acids are a further source of hydrogen ions. It should also be noticed that with the progress of decomposition the source of H^+-ions is no longer restricted to water.

PROCESSES OF DECOMPOSITION

Hydrolysis and hydration

The ions H^+ and OH^- are chemically active and they react readily with ionized elements in the minerals. Such ionized elements, particularly metal ions in silicate minerals of igneous and certain metamorphic rocks, are always available because of broken bonds and unoccupied positions in the crystal lattice. The reaction of hydrogen (H^+) or hydroxyl (OH^-) ions is known as *hydrolysis* where the hydrogen ion is particularly important because, due to its very small volume, it can enter the crystal lattice more easily than any other ion. Since these and all other reactions during decomposition are subject to the rules of chemical equilibrium, they can only occur if a component is added to, or removed from the system. As a consequence, when the hydrogen ion with its strong electrical field enters the openings between the oxygen atoms of a crystal lattice, it forces the larger metal ions held in these openings out of the system. The changed or newly forming minerals increase in volume and this leads to the disruption of the rock and mineral structures and to the creation of more broken bonds. Eventually, large enough spaces develop to allow whole water-molecules to enter: *hydration* takes place followed by a further increase in the volume of the affected minerals.

Hydration is also a reaction between ions situated in the surface of rocks and of minerals. These ions attract the suitably polarized side of water-molecules, e.g. ions of ferrous iron (Fe^{++}) tend to hold the negatively charged oxygen side, and ions of oxygen (O^{--}) the positively charged hydrogen side of a water-molecule. These bonds can be quite strong. Surface hydration often preceeds hydrolysis: the water so held superficially provides the hydrogen ions which eventually force their way into the crystal lattice.

Hydrolysis and hydration are bound to the presence of water and, therefore, the more water available to act on rocks of a suitable composition, the more effective they are.

Oxidation and reduction

Oxidation is the process of combining with oxygen. The term is, however, also used more generally to indicate an increase in the positive, or a decrease in the negative valences of a chemical element. These changes usually lead to the reaction of such an element with oxygen. Such a change of valences is the particular property of certain metal ions which may 'jump' from a lower to a higher state of oxidation at such a change. Although not the only one, iron is the most commonly occurring metal of this type. Iron is bivalent in the ferrous state (FeO) and trivalent in the ferric state (Fe_2O_3). In most iron-containing silicate minerals, the metal occurs in the ferrous state, but when it comes into contact with the atmosphere it changes easily into the more highly oxidized and, under surface conditions, more stable ferric state. The presence of water enhances oxidation and the iron first gathers OH^--ions to form the unstable hydroxide $Fe(OH)_3$ which is then hydrated to the stable $Fe_2O_3.nH_2O$. There are two modifications of ferric iron, the anhydrous Fe_2O_3 (the mineral haematite) and the hydrated $Fe_2O_3.nH_2O$ (the mineral limonite) where n usually has the value 2. These iron compounds have a vermilion red colour in the anhydrous, and a brown colour in the hydrated state.

The predominant colour of soils or the weathered surface of rocks depends much on climate and it tends towards brown where N is less than 5, and toward red where N is more than 10, with transitional conditions where N is between 5 and 10. The climatic environment also has an influence on the depth of oxidation in a rock: in the presence of sufficient water, i.e. again where N is less than 5, the effect of oxidation can be deep, in the order of metres in the case of coarse-grained rocks, in contrast to areas where N is more than 10 and where all available moisture may be limited to some dew at night leading only to the formation of an oxidized film a few millimetres thick.

The opposite of oxidation is *reduction* in which oxygen is lost. In nature, again, iron is the most common metal subject to reduction, in which case it changes from the ferric to the ferrous state. Reduction occurs under conditions where the access of oxygen is restricted, e.g. in water-logged soils, below the water-table or in the presence of carbon-containing material like humic acid. The colour of soils or weathered rocks where iron is in the ferrous state is grey to dark grey, green or blue and, for instance, the dark colour of the black clay of the Highveld must be ascribed to the presence of ferrous iron.

The shiny black colour of the oxide film on many rocks in an arid environment where N is more than 10, must not be confused with the dark grey and, when wet, nearly black colour of the Highveld clay in areas where N is between 2 and 5. The black colours of the surface film on rocks in desert areas, known as *desert varnish*, is caused by manganese oxide (MnO). Manganese is lost where sufficient moisture is available because manganese oxide is more soluble than the iron oxide FeO. Manganese oxide is retained, however, in a dry environment and a small quantity suffices to change the colour of the oxide film from red to black.

Carbonation

When suitable ions are available, particularly those of calcium, but also magnesium, iron and others, their reaction with the carbon dioxide of the air, dissolved in rain water, causes carbonation, i.e. compounds of the carbonate ion CO_3^{--} and metal ions. The degree of carbonation depends on the availability of suitable ions and the retention of the usually soluble carbonates in the weathering rock or the soil. Therefore, although water is required for the formation of carbonates, it must not be present in such quantities that the carbonates are dissolved and removed. This is probably the reason why the formation of certain carbonates, especially calcrete, is most pronounced where N is between 5 and 10.

More information may be obtained from text-books on geology, petrology, geochemistry and pure chemistry such as **Glasstone** (1956), **Dapples** (1959), **Birkeland** (1974) and, with particular reference to Southern Africa, **Harmse** (1974) or the relevant chapters of the very comprehensive Handbook on Geochemistry (**Wedepohl**, 1969-).

Desilicification

Desilicification, the loss of silica (SiO_2) from primary or secondary silicate minerals, is occasionally considered another process of decomposition. This is not quite correct, however, because desilicification is the result of the processes described above rather than a process in itself.

DECOMPOSITION OF ROCKS

Although mostly restricted to crystalline rocks and a particular environment, decomposition is a form of weathering which requires much more attention than disintegration. The mechanism of decomposition has always been called 'chemical weathering' but 'mineral transformation', which is, of course, associated with certain chemical processes, would be a better description.

The essential feature of crystalline rocks is their composition of minerals which have formed and can exist under conditions of temperature and pressure that differ greatly from those prevailing on the surface of the earth. This applies in particular to the igneous and certain metamorphic rocks while only a few of the sedimentary rocks, depending on their mode of origin, may occasionally contain components which make them subject to decomposition. Igneous rocks are more strongly affected by decomposition than metamorphic ones. This is obvious if one remembers that igneous rocks develop from a hot magma, which means a more or less completely molten material. Depending on the depth at which this magma cools, which can vary from more than a hundred kilometres below to right on the surface, the pressures involved vary from a hundred kPa to several MPa. In rock metamorphism the temperatures involved are lower than those involved in the formation of igneous rocks. This is obvious because, if the temperatures which lead to igneous rocks were attained in metamorphism, the rock would melt. Re-melting, a process known as anatexis, may occur in the course of extreme metamorphism and very special types of igneous or metamorphic rocks are the result. Melting temperatures are also not attained in thermal metamorphism where rocks next to an intruding magma are affected by high temperatures but without an increase in pressure.

The minerals which are most susceptible to decomposition are therefore amongst those which are more typically found in igneous than metamorphic rocks. This is a very generalized statement, however, because although there are minerals which are typically formed during metamorphism, e.g. garnet or tourmaline, most rock-forming minerals possess varieties which form under the conditions of metamorphism, dynamic, regional as well as thermal. This is the reason that rocks, whether igneous or metamorphic, where quartz, feldspar, mica, amphibole, pyroxene or olivine are the predominating primary minerals, are placed in the common class of crystalline rocks. The way in which these rocks decompose and the road building properties of their decomposition products are so similar that the minor variations, which stem from the structural and compositional differences between the igneous and metamorphic members of the same mineral type, are of no relevance to their road performance.

Only a limited number of rock types are normally subject to decomposition. The particular geological setting of Southern Africa, however, is the reason for the importance which must be attached to decomposition if the road building quality of weathered rocks needs to be assessed. Crystalline rocks predominate or are among the most important road building materials in considerably more than half the area of the sub-continent. They are mainly acid, mostly granite in Rhodesia and South-West Africa, and in the Transvaal, the North-Western and South-Western Cape Province and along the coast of Natal, and basic, mostly dolerite in the vast areas where rocks of the Karoo System occur, and others in a number of other geological systems, e.g. the norite of the Transvaal.

Crystalline rocks only decompose in a suitable environment which provides sufficient water to initiate the chemical and mineralogical alterations which make the decomposable primary minerals change into secondary ones. This environment has been defined by an N-value of less than 5 (**Weinert**, 1964, 1965, 1974), although where water can collect locally, e.g. in surface depressions or in cracks in the rock, decomposition may also occur in areas where N is between 5 and 10. This means that decomposition is restricted not only to certain types of rock but also to certain regions: only in part of Southern Africa is the environment such that decomposition is dominant over disintegration. Nevertheless, decomposition retains its particular importance for Southern Africa because of the widespread

occurrence of rocks which can decompose and the fact that virtually all highly developed areas of the sub-continent are in an environment where N is less than 5. There are a few exceptions, like Kimberley, based mainly on mineral resources, but all large-scale developed areas, agricultural as well as industrial, and the consequently higher density of population, all of them heavily dependent on water, are confined to an environment where N is less than 5.

The logical consequence is that these areas also have the densest road network and that the call on road building materials is greatest here with the need to use whatever is offered by nature. Local deviations from the present conditions will occur in future with the advance of irrigation schemes, particularly in the Orange River valley.

THE DECOMPOSING MINERALS

Quartz, the felspars, the micas, amphibole, pyroxene and olivine are the minerals which characterize crystalline rocks, i.e. the rocks which are most likely to decompose in a suitable environment.

The six minerals, or groups of minerals, are given roughly in the order of stability, beginning with the most stable, quartz, and ending with the very sensitive olivine. Simultaneously, this sequence also represents an increasing remoteness of the conditions of origin of these minerals from the conditions on the surface of the earth, which in turn represents approximately the sequence of origin: olivine was formed very early in the cooling process of a magma under conditions that differed most from those at the surface, and quartz was formed last and under conditions much more similar to those at the surface. Since the conditions of origin have an effect on the degree of instability of the minerals under surface conditions, this sequence also indicates the susceptibility of the minerals to decomposition.

There is a certain parallel in the degree of resistance of these minerals to decomposition. The parallel which has been postulated by **Goldich** (1938) follows from the considerable compositional and structural differences among the members of the feldspar group of primary rock-forming minerals. Generally speaking, the more calcium and the less silica feldspars contain, and the more markedly tricline their crystallographic structure is, i.e. the more they are of the anorthite type, the more easily they decompose. There is a similar difference in the resistance to decomposition between the members of the mica group, muscovite and biotite.

The arrangement of the important primary rock-forming minerals according to susceptibility to decomposition, as proposed by **Goldich** (1938), is given in Table 2 in the form presented by **Brink** and **Williams** (1964):

It should be noted that there are two groups of minerals whose members occupy different positions in the Table above, and further that the above minerals are all 'primary', i.e. they were formed during the cooling of a magma. It may be possible to add more minerals to this arrangement even if they are not necessarily 'primary', for instance the sulphide minerals, pyrite, chalcopyrite and others whose position in the arrangement would probably be in the vicinity of olivine, but which do not decompose to clay minerals.

Micas

The *mica* group has two principal members: muscovite, the light-coloured mica, and biotite, the dark-coloured mica. After quartz, *muscovite* is the mineral which is most resistant to decomposition. For a long period, muscovite only disintegrates until eventually the by that time small mineral pieces change, through stages of hydration, into kaolinite clay. *Biotite*, which contains many more bases, particularly magnesium and iron besides potassium which is common to all micas, decomposes more easily than muscovite. Illite, a clay mineral of that ill-defined group 'hydromica' which is often composed of alternating sheets of montmorillonite and illite, which latter have developed from biotite by the loss of magnesium and iron, is formed first and it remains the end product of decomposition unless conditions are such, especially when N is less than 2, that part of the illite, after changing through intermediate stages, eventually becomes kaolinite. The illite-montmorillonite mixed-layer clay mineral has engineering properties which are similar to those of montmorillonite.

Feldspars

The *feldspars* are the most important group of rock-forming minerals. The two principal types are orthoclase, the potassium feldspar, and plagioclase, the sodium-calcium feldspar. *Orthoclase* is a resistant mineral (only muscovite and quartz are more resistant) which decomposes slowly into kaolinite clay wherever

TABLE 2
Resistance to decomposition of the primary rock-forming minerals
(After Goldich (1938) and Brink and Williams (1964))

OLIVINE (serpentine)

           Ca-PLAGIOCLASE (montmorillonite clay)

PYROXENE (montmorillonite clay)

           Ca-Na-PLAGIOCLASE (montmorillonite clay)

AMPHIBOLE (montmorillonite clay)

           Na-PLAGIOCLASE (montmorillonite clay)

BIOTITE (silt, illite clay)

           ORTHOCLASE (kaolinite clay)

MUSCOVITE (silt)

           QUARTZ (sand)

N is less than 5. The *plagioclases* form a complete series of mixtures of the two end-members albite, the sodium feldspar, and anorthite, the calcium feldspar. They decompose easily, the anorthite more so than the albite, when N is less than 5, and montmorillonite clay is the final weathering product where N is between 2 and 5, while part of the montmorillonite changes into kaolinite when N is less than 2.

Mafic minerals

Amphibole, *pyroxene* and *olivine* have certain similarities. They are all dark-coloured minerals, being silicates of several metals, especially iron, magnesium and calcium, and they form the group of mafic minerals to which biotite may also be added. In addition to the above components, amphibole contains the hydroxide ion (OH^-) and olivine is poorer in silica (SiO_2) than the others. When decomposing, they all become montmorillonite eventually, olivine after having initially changed into serpentine and amphibole and pyroxene into chlorite, where N is less than 5: they all partially change into kaolinite again where N is less than 2.

Quartz

Quartz decomposes only under such exceptional conditions that they can be neglected for the purpose of road construction. The amorphous silica which is often found in residual soils and from which secondary quartz may develop under certain conditions, is derived from the silica contained in and released by the other decomposing silicate minerals. The great stability of quartz is the reason that this mineral is of much greater importance and much more widely present in sedimentary, than in crystalline rocks.

THE END PRODUCTS OF DECOMPOSITION

All minerals which are likely to decompose under suitable conditions eventually change into some type of clay. The clay is either montmorillonite or kaolinite, depending on the parent rock and the environment, although illite and other secondary minerals may predominate in certain intermediate stages of decomposition. The type of parent rock and different environments are not independent parameters in weathering, and there is therefore a systematic trend in the type of minerals which develop during decomposition.

When considering the use of decomposing rocks in road construction, it is sufficient to distinguish between *acid* and *basic* crystalline rocks, dividing the intermediate types between them in such a way that the orthoclase-containing rocks, e.g. syenite, are considered as 'acid' and the plagioclase-containing ones, e.g. diorite, as 'basic' (**Weinert**, 1968). There are of course transitions whose inclusion in one group or the other should be decided according to the type of clay mineral, kaolinite or montmorillonite, which most strongly affects the road building properties of the decomposed rock.

The terms *acid* and *basic*, as used in the classification of rocks, must not be understood in the purely chemical sense although they have been derived from chemistry. Nowadays, these terms are used quite loosely, denoting rocks composed of more or less than 65 per cent of silica respectively. The high percentage of silica leads to the formation of quartz as a principal component of crystalline rocks, especially the igneous types. Both terms, acid and basic, are increasingly going out of use in petrology, being replaced by grades of saturation with silica. They will be retained in this context, however, mainly because they are generally known and understood by South African road enigneers. The terms will be used in the sense that 'acid' refers to rocks which contain free quartz in such a quantity that the mineral can be detected easily or, if quartz is scarce or absent as in syenite, that it refers to rocks from which kaolinite develops directly during decomposition, in contrast to the essentially quartz-free 'basic' rocks which decompose into montmorillonite clay.

An acid crystalline rock contains quartz as one of its principal minerals and this means that quartz can be recognised at a glance. This is usually the case if there is more than 10 per cent of quartz present. In granite and its metamorphic derivative, orthogneiss, which are the most typical and the most widely occurring representatives of this group, quartz makes up about 30 per cent of the volume of the rock, orthoclase feldspar about 60 per cent and the remaining 10 per cent is either mica or amphibole or both plus a few accessories. If such a rock decomposes, the first changes are brought about by hydrolysis, oxidation and hydration of the component minerals, except quartz, followed by a rearrangement of atoms in the crystal lattices. This rearrangement affects in particular the silica chains of the feldspars, amphiboles, pyroxenes and, of course only in basic rocks, olivines which change into silica layers. In the micas, which are primary layer silicates, the metal ions between the silica layers are gradually replaced by hydroxide. Clay-type minerals which are known by the loosely connecting name of hydromica develop. Continuing decomposition leads to kaolinite clay, which develops from the orthoclase, and from biotite and amphibole to montmorillonite initially, eventually also changing into kaolinite. The high percentage of orthoclase in acid rocks leads to quartz-containing kaolinitic clay soils as final decomposition products.

Quartz is absent in the basic crystalline rocks and the predominant minerals are plagioclase, representing the feldspars, and mafic minerals. Again, decomposition starts with hydrolysis, oxidation and hydration and hydromicas or chlorite develop. Montmorillonite is the next stage of development and, under favourable conditions, but only in the top half-metre or so of the weathering profile, this montmorillonite may change into kaolinite while montmorillonite remains the predominant clay mineral in the lower parts of this profile.

INFLUENCE OF ENVIRONMENT

The principal environmental influence on the mode of weathering is the local climate but this influence can be modified to a certain degree by the local topography, and a few other factors.

The influence of climate on weathering is expressed by the N-value and only four values, all of them integers, are significant (see Figure 8).

The most important value is N=5. Where N is more than 5 disintegration, and where N is less than 5 decomposition is dominant.

Where N is more than 10, no significant weathering profile develops and there is only a thin layer, usually less than half a metre thick, of coarse gravel obtained from the disintegrating rock. Weathered rock for road construction in these areas can often only be obtained from borrow 'areas' (*Plates 9 and 10*), i.e. the required quantity is obtained by ripping large areas to a shallow depth. The gravel contains few fines because most fines which might develop are washed away by the occasional showers or blown out by wind. Some binder must therefore always be added to this wholly cohesionless material.

Where N is between 5 and 10, disintegration still predominates but a deeper weathering profile is usually present and road building materials can be obtained from borrow pits (*Plate 11*). Although disintegration is the principal mode of weathering, a limited quantity of secondary minerals may develop, being mostly illite, which may even exhibit some plasticity. Their quantity is, however, hardly ever sufficient to act as a natural binder and the addition of another binder is therefore always required.

Where N is between 2 and 5, decomposition becomes the predominant form of weathering of crystalline rocks, and weathering profiles from fresh rock to residual soil now develop. The clay component in such a profile is kaolinite if the profile has developed from an acid crystalline rock, and montmorillonite, and consequently expansive, if derived from a basic crystalline rock. The depth of weathering increases from N=5 towards N=2 and very often considerable quantities of overburden have to be removed to obtain material of the required quality.

Where N drops below 2 and, consequently, where much water is available, montmorillonite in the top portion of the weathering profile on basic crystalline rocks, i.e. in the residual soil, loses its bases and one silicate layer and changes into kaolinite. In Southern Africa, this change never affects more than the first metre of the profile, usually only half of it, and montmorillonite remains the final decomposition product below. In spite of the cover of kaolinitic clay, therefore, decomposed basic crystalline rocks do not lose their expansive properties. In acid crystalline rocks, no such fundamental change occurs because kaolinite is the predominant clay mineral wherever these rocks decompose, i.e. where N is less than 5. Montmorillonite in acid crystalline rocks, which may have developed from the little biotite or amphibole present, is subject to the same change if N is less than 2 as is montmorillonite in decomposed basic crystalline rocks.

Where N is less than 1, silica is often lost as well and only sesquioxides of aluminium (Al_2O_3) or iron (Fe_2O_3), usually in the hydrated state, are left behind. Hydrated aluminium sesquioxide minerals like diaspore, boehmite, bauxite and gibbsite are the major constituents of these truly lateritic soils owing to the predominance of aluminium over iron in most crystalline rocks (**Harrison**, 1933; **Neves**, *et al*, 1969; **West** and **Dumbleton**, 1970; **Valeton**, 1972). The rocks are decomposed to great depths, 50 metres and more, where N is less than 1. The weathering profile which results from the decomposition of crystalline rocks is retained as before but the stages at which different types of clay mineral predominate, move downwards. This means that the decomposed material on acid crystalline rocks is wholly kaolinite, and that on basic crystalline rocks the soil is kaolinite on top and montmorillonitic lower down the profile, before the fresh rock is reached.

In an environment where N is more than 10, no secondary minerals develop, i.e. the primary minerals of crystalline rocks do not change and the rocks only disintegrate. Where N is between 5 and 10, disintegration is the predominant form of weathering of crystalline rocks, although various types of hydromica may develop on a limited scale but the physical break-down of the rock still determines the road building properties of crystalline rocks. Where N is less than 5, crystalline rocks decompose and, depending on the situation of the site on either acid or basic crystalline rocks, and in an environment of either N less than 2 or N between 2 and 5, kaolinite, montmorillonite or both are the principal secondary minerals. Quartz does not decompose and the final decomposition products depend on the other decomposing minerals and the climatic environment, as shown in Table 3.

Where N is less than 1, hydrated sesquioxides of mostly aluminium and occasionally iron often take the place of kaolinites.

By combining Tables 2 and 3, the relative susceptibility of rocks to decomposition can be estimated. *Quartzite* and *vein quartz*, both composed of quartz only,

TABLE 3
Decomposition products of rock-forming primary minerals

N-value	Minerals		
	Quartz	Feldspar Orthoclase ← → Plagioclase Mica Muscovite ← → Biotite	Amphibole Pyroxene Olivine
N > 10	Quartz	(no change)	(no change)
N 5-10	Quartz	(Hydromica)	(Hydromica)
N 2-5	Quartz	Kaolinite	Montmorillonite
	Quartz	Kaolinite	Kaolinite / Montmorillonite

do not decompose and a basic crystalline rock, such as *dolerite* or *norite*, composed of plagioclase and pyroxene, decomposes more easily and gives less desirable road construction material than an acid crystalline rock like *granite* which is composed of quartz, orthoclase and some mica or amphibole.

INFLUENCE OF RELIEF

Topographical features have a modifying influence. Ideally, the above conditions of decomposition apply to flat, possibly water-logged areas. Such areas can be defined as having a gradient of less than 5°. If the gradient increases, run-off and also the circulation of water within the soil increase. This means that water which is saturated with mobilized and dissolved material is easily removed and replenished by unsaturated water. Therefore, if the circulation of water increases, montmorillonite on basic crystalline rocks also starts to change into kaolinite in areas where N is between 2 and 5. This change is first noticeable as a thin band of kaolinite, about 50 mm thick, overlying the montmorillonite. It is often overlooked. If the gradient exceeds 15°, however, the thickness of the kaolinite layer increases noticeably. At this gradient, the loss of weathering residues, now on all types of rock, also becomes noticeable and the thickness of the weathered layer decreases, i.e. the slightly weathered or unweathered rock occurs nearer to the surface than in flat country under the same conditions of N. Where the gradient approaches 30°, very little loose residual soil is retained and at an angle of 60° and more only the bare rock is exposed.

The above observations on the modifying effect of the topography on the mode of decomposition and the thickness of the weathered layer in a given N-value environment have, of course, an important bearing on the selection and situation of borrow pits. The chances of gaining access to a material of desired quality, if it is covered by overburden too thick to make production economical, are greater if it can be obtained on a slope, particularly if the gradient is more than 5°. It is obvious that better quality material will be closer to the surface as the gradient approaches 15° but, since a gradient of 15° may already pose serious problems in regard to the access, the optimum situation of a borrow pit must be dictated by the value of the required material and the economics of its production.

INFLUENCE OF OTHER ENVIRONMENTAL FACTORS
Organisms

Plants and animals promote weathering in different ways but their overall effect is less pronounced than that of climate and topography. The roots of plants and digging animals may open cracks and allow easier access of water, and decaying plant material may supply humic acid to the soil water, thus increasing its chemical activity. These influences largely cease, however, if weathering material is removed from its site and placed in a road layer.

Bacteria may hasten the decomposition of sulphide minerals, e.g. pyrite, and this process need not come to an end after the laying of a material which contains such minerals. Provided sufficient oxygen is available in the layers of a pavement,

these minerals may continue decomposing, eventually changing into soluble salts after passing through a stage where free sulphuric acid is formed. Since the amount of oxygen available under a bituminous surfacing may vary, the intensity of this process will also be variable. For more detail, reference should be made to Chapter 10.

Time

Since weathering is a process of change, time is a factor of some influence. Time attains a very great effect if measured in geological orders of magnitude and when considering the total change from a fresh rock to a residual soil. The usual length of the design life of a road structure is of course negligible compared with geological times of any significance and a fresh rock, used for road construction, can never change into a residual soil during this geologically very short period. The design life of a road can, however, be seriously affected if the weathering condition of a construction material is close to the point where this material would no longer be suitable.

The rate of weathering increases with its progress. It starts slowly on the fresh rock although, in a climatic environment where N is less than 5, the design life of a road is long enough to show the appearance of an unmistakable stain of oxidized material on the surface of such a rock. The more weathered a natural road building material is at the time of laying, the more pronounced such noticeable changes are during the design life of a road. Since a material continues weathering in the structural layers of a road (**Clauss**, 1967), its condition may be such that the design life of a road or an even shorter period may suffice to bring the material below the specified standard; material failure is the consequence. The determination of the stage of weathering of a natural road building material should therefore be the first step in the assessment of the quality of such materials (**Weinert**, 1964) (see Chapter 7).

REVIEW OF WEATHERING

Decomposition is the more complex process of weathering and, although it mostly applies only to the crystalline rocks and those members of the group of diamictites, e.g. tillite or greywacke (see Chapter 8), which contain a large quantity of fragments of crystalline rocks or unweathered primary minerals, it has deserved this extensive discussion. Decomposing rocks show a much greater variability, petrologically as well as regards their road building properties, than disintegrating rocks. Besides the variation of the physical properties, such as strength, plasticity and others, of decomposing crystalline rocks, it is their durability in particular which is at the root of the numerous problems experienced with these rocks when they are used for road construction. Disintegrating rocks are durable materials in the sense that, once their properties have been determined and found to be satisfactory either in the natural state or after some treatment, they can safely be used: they will not change to such an extent as to cause failures. Decomposing rocks, however, continue decomposing, mostly at an accelerated rate, after having been

placed in a road structure and it is therefore necessary to ascertain whether or not the state of decomposition of these rocks is such that the continued decomposition will not render them unsuitable prior to the expiry of the structural design life of the road. Although only crystalline rocks decompose, their occurrence nearly everywhere in Southern Africa makes them the most commonly used road building materials and their properties must therefore be understood better than those of any other major group of road building materials.

CLASSIFICATION OF WEATHERED MATERIALS

It has been found practicable to subdivide a weathering profile into four stages. Many such classifications, mostly consisting of more than four stages, have been developed (**Knill** and **Jones**; 1965; **Geological Society of London, Engineering Group Working Party**, 1970; **Fookes** *et al.*, 1971; **ASCE Task Committee for Foundation Design Manual**, 1972). The greater the number of subdivisions, the more detail can be introduced into such a classification system, but the more detail is also required for making a decision on the position of an individual sample within such a classification system. This makes detailed classification systems, which eventually always involve determinations that can only be done in a laboratory, unsuitable for use in the field. The easiest grouping is of course a twofold one based on the reply of either 'yes' or 'no' to some qualifying question. However, this would obviously not suffice for a decision on the suitability of a material for road building purposes. A subdivision in three stages which can still be applied without involving sophisticated investigations or tests, contains the chance that one of the three groups develops into a 'miscellaneous box' which eventually houses the majority of all cases. The four-fold grouping requires a definite decision on each individual step but, since it could be considered as being based on a two times twofold subdivision, it can still operate without complicated investigations and is therefore applicable in the field.

This fourfold subdivision of the different types of weathering profiles is given in Table 4.

TABLE 4
Classification of weathering

GENERAL	MECHANISM OF WEATHERING	
	Physical	*Chemical*
Residual soil	Residual gravel Residual sand	Residual clay soil
Highly weathered	Highly disintegrated	Highly decomposed
Weathered	Disintegrated	Decomposed
Fresh rock	Fresh rock	Fresh rock

Fresh is that condition in which a rock, after being deprived of the overburden, and being exposed to the atmosphere or hydrosphere, starts weathering. Crystalline rocks are regarded as fresh for the purpose of road construction as long as they contain less than 15 per cent of secondary minerals (**Knight** and **Knight**, 1948; **Weinert**, 1964). Fresh rock which satisfies the strength and, where necessary, other requirements is the source of all surfacing and concrete aggregate and of most stone for crushed stone bases.

Weathered is the freshest stage where the effects of weathering have become noticeable. The rock has started to break down into pieces although the majority of these pieces still have a diameter of more than 50 mm. The size and massiveness of most of these rock pieces, which are often big blocks, make excavation with a pick difficult if not impossible, and blasting is still required in many cases. Materials for natural gravel bases and occasionally also for subbases, particularly in the case of major roads, are in this state of weathering and care must be taken if the rock is decomposing.

Disintegrated rocks are severely cracked and along the cracks they are usually covered by an oxidized surface layer which, due to the iron oxide content, is mostly reddish or brownish coloured and, where N is more than 10 and where manganese oxide is involved as well, the outside of the rocks may attain the dark brown to black, glistering stain of desert varnish. These oxide layers can be remarkably thick, giving the rock a brown to red skin, and are in strong optical contrast to the freshness of the interior of the rock pieces.

The stage *decomposed* refers of course only to crystalline rocks and certain diamictites and it occurs only in an environment of N less than 5. In addition to the physical properties related to cracks, and the general size of the broken pieces of rock which are in accordance with the general condition 'weathered', mineralogical alterations are unmistakable. Most primary minerals, except quartz, have attained a dull colour. Even in the centre of the rock blocks or pieces, which always appears fresher than the outside, the minerals no longer flicker markedly when turned in sunlight as they do in a fresh rock. Between 15 and 30 per cent of the primary minerals have at this stage changed into secondary minerals which can be recognized by the lighter, mostly brownish, dull colour on the face and inside the rock pieces, and by the brittleness and, in certain cases, softness of the mineral grains: some can be removed easily with a needle.

Highly weathered and similarly, 'highly disintegrated' and 'highly decomposed', replaces the former 'badly weathered', 'badly disintegrated' and 'badly decomposed' because weathering does not justify qualification into 'good' and 'bad'. In the highly weathered stage, the diameter of the rock pieces is mostly less than 50 mm and down to gravel and sand size, for example the 'sugar gravel' of the road builder. The material has thus reached that state where it is still 'rock' petrologically but where it must be treated like a 'soil' in engineering. Excavations can be done by means of mechanical shovels, bulldozing or even by hand and blasting is not requir-

ed. Highly weathered materials are used for the subbase of many roads, for gravel wearing courses and for selected subgrades and fills.

The small size of the rock pieces is the principal criterion for the classification of a rock as *highly disintegrated*. Surface staining by oxides, or desert varnish where N is more than 10, is very pronounced. In spite of this stain, the majority of the minerals are still fresh as can be noticed by the flicker when crushed pieces are turned in sunlight or by the bright colour if viewed in a microscopic slide under crossed nicols. A certain number of secondary minerals, generally of the illite type, may have developed in crystalline rocks where N is between 5 and 10. The total number of these secondary minerals, however, always remains low because they develop very slowly in this environment and are washed away by the occasional heavy showers in this climate.

The range of conditions among *highly decomposed* crystalline rocks or in certain cases diamictites, is greater than their equivalent range in the progress of disintegration. Highly decomposed rocks contain more than 30 per cent of secondary minerals, the majority of which are kaolinite or montmorillonite depending on the parent rock. These secondary minerals are present throughout the rock which has lost its original colour completely; the rock has attained a light brownish, yellowish or greyish colour even in the centre of bigger-sized lumps. Relics of fresh, hard minerals which are still retained in such lumps, can easily be rubbed off with a finger and the consistency of the lumps is usually so poor that they are broken down by rain. The shape of the crystals of the original primary minerals is often retained although the whole mineral has changed into whitish clay. Highly decomposed rocks in an advanced stage of decomposition have become so soft that they can be excavated with a spade and, although they are still rock in the petrological sense, they must be treated like soil in engineering: e.g. in testing, the normal soil tests such as Atterberg limits and others apply.

Residual soil is that state which obtains when the weathering rock has attained equilibrium with the prevailing environmental conditions. Only occasionally can the residual material be used in road construction, mainly for gravel wearing courses or selected subgrades, but in most cases it is only the natural ground on which the road structure rests.

Where the rock has disintegrated, the residual material is either *residual gravel* or *residual sand* (*Plate 12*), depending largely on the size of the minerals in the parent rock, or, if the parent rock was very argillaceous, a residual clay. Except for the clay, these soils are easily workable, durable materials particularly for the lower layers of the pavement or for gravel riding surfaces. Their only disadvantage is the frequent lack of natural binder. Residual clay formed by disintegration, which does not occur often, is quite useless.

Decomposing rocks change into a *residual clay soil* which is predominantly reddish, sandy to silty, kaolinitic clay if the parent rock was acidic, and which is dark grey, almost pure montmorillonite clay if derived from a basic parent rock (*Plate 13*). Neither of them is of use as a road building material although kaolinite can serve as a natural binder if occurring in, or added to, an otherwise cohesionless material. Kaolinite is less plastic than montmorillonite and it does not expand

noticeably. This is not the case with montmorillonite which owes its high expansiveness to the large quantity of water which can enter the crystal lattice. All clay soils have a low bearing capacity, the soaked CBR occasionally being little more than 5, and this must be allowed for in the overall structure of the road when these clays form the natural subgrade. If this is done, a road can be built on kaolinite clay, e.g. soil residual from granite, without endangering the structure. Whenever possible, however, natural ground of montmorillonite clay, e.g. the 'black clay' derived from dolerite or norite in environments whose N-value is between 2 and 5, should be avoided because of the expansiveness of these soils.

In areas where N is between 2 and 5, lime nodules may develop in highly decomposed basic crystalline rocks and in the residual soil cover (*Plate 14*). Such nodules must of course also be regarded as secondary minerals because they have developed in situ from the calcium released during the decomposition of primary minerals. They may have accumulated locally to form nodular calcrete in which case they may be welcome sources of subbase material.

The general shortage in Southern Africa of transported soils and gravels which could be suitable materials for road construction, necessarily calls for the use of weathered rocks as sources of road gravels. Weathering produces very varying materials because its effect depends on the interaction of climate, topography, organisms and time in the different parent rocks. Weathering is further not arrested by the removal of a material from its natural site and placement in a road structure, and weathering materials therefore continue weathering in the road layers, often at an increased rate. Weathering rocks are consequently not unchangeable road building materials, and the assessment of the degree of weathering at the time of selection is of prime importance to ensure the use of a material which is sufficiently durable to remain of acceptable quality during the whole design life of the road.

CHAPTER 5
PROPERTIES OF NATURAL ROAD CONSTRUCTION MATERIALS

GENERAL

The basic source of natural road building materials is rock, in contrast to soil which is secondary in nature being derived from the weathering of rocks. This requires the discussion of some of the features of rocks which determine their suitability as road construction materials.

The property of a rock as a construction material depends primarily on its mineral composition and secondarily on the size, shape and arrangement of and bond between the minerals, i.e. the texture of the rock, while the larger-scale structural features are less important although they must not be neglected entirely since, for example, they naturally affect the excavation characteristics.

MINERALS

The total number of minerals is very great and, theoretically, the engineering quality of each mineral in a rock should be assessed. The properties of the rock concerned could then be deduced from the interaction of the properties of its mineral components. Obviously such an approach would be impractical and, fortunately, the majority of all minerals occur in rocks only as accessories and the remaining minerals can be grouped together so that a number of them have sufficiently similar properties to be treated as entities. These latter minerals of which there are between a dozen and a score are known as the rock-forming minerals.

Most rock-forming minerals are silicates which means that silicon dioxide (SiO_2 = silica), is the essential part of their composition. Others are carbonates, i.e. minerals containing carbon dioxide (CO_2), and certain sulphur-containing minerals, such as pyrites, may also attain some importance.

The silica, if contained in a cooling magma from which primary minerals develop, first enters into compounds with other constituents of this magma. If there is more silica than these other constituents can use up during the crystallization process and the formation of minerals, the remaining silica crystallizes into quartz, SiO_2, the mineral composed of pure silica. It is important to notice that the crystallization of silica into quartz takes place toward the end of the whole cooling process of a magma when the temperature and also the internal pressure have decreased considerably. This means that quartz is formed in a temperature-pressure equilibrium which differs less from the conditions near or on the surface of the earth than do the conditions of formation of most other primary minerals. These other primary minerals are therefore inclined to change into secondary minerals once this equilibrium has been disturbed.

The rock-forming minerals which must be considered when assessing the suitability of natural road building materials are those shown in Table 5.

TABLE 5
Rock-forming minerals

MINERAL GROUP	IMPORTANT MEMBERS	CHEMICAL COMPOSITION	REMARKS
Quartz		Pure silica: SiO_2	
Feldspar	Orthoclase	K, Al – silicate	
	Plagioclase	Na, Ca, Al – silicate	
Mica	Muscovite	K, Al – silicate with hydroxide	Sheet lattice
	Biotite	K, Mg, Fe, Al – silicate with hydroxide	Sheet lattice
Pyroxene	Augite Hypersthene	Fe, Mg, Ca, Na – silicate	
Amphibole	Hornblende	Fe, Mg, Ca, Na – silicate with hydroxide	
Olivine		Fe, Mg – silicate	Low silica content
Calcite		Ca – carbonate: $CaCO_3$	
Dolomite		Ca, Mg – carbonate: $CaMg(CO_3)_2$	
Gypsum		Ca – sulphate with $2H_2O$	
Sulphate salts		Sulphates of Na, Mg and other metals with varying H_2O	
Rock salt		Na – chloride (NaCl)	
Clay minerals	Kaolinite	Al – silicate with $2H_2O$	Sheet lattice
	Montmorillonite Illite	Na, Ca, Al – silicate with Mg or Fe and nH_2O	Sheet lattice (expansive)
	Attapulgite	Mg, Al – silicate with H_2O	Chain lattice
Opal		Hydrated amorphous silica	
Pyrite	Iron pyrite	Fe – sulphide	Gold colour
	Marcasite	Fe – sulphide	Silver colour
	Chalcopyrite	Fe, Cu – sulphide	

The minerals quartz, feldspar, mica, pyroxene, amphibole and olivine are characteristic components of igneous and certain metamorphic rocks and the rocks which are composed of these minerals will be called crystalline rocks. These minerals, except quartz, are liable to decompose, i.e. they may change by chemical weathering and give rise to the development of secondary minerals, particularly clay.

The minerals calcite, dolomite, gypsum and other sulphate salts, rock salt and the clay minerals are characteristic of sedimentary rocks and some of them also occur in certain metamorphic rocks. Again, quartz may be added to this group since, because of its resistance to weathering, it is a very important constituent of sedimentary rocks. These minerals are the end products of some form of weathering, especially decomposition, and they therefore do not decompose further. It should be realized in this connection that the carbonate minerals, calcite and dolomite, do not decompose in the strict sense: they dissolve in water, as do all other salts, and they are precipitated again as carbonate minerals, although minor mineralogical changes may occur, such as the reprecipitation of dolomite as calcite (see Chapter 4).

Pyrites and opal can occur in crystalline and sedimentary rocks. The pyrites decompose easily and fast once they are exposed to air and water, and the decomposition is aggravated if, in addition, bacterial action occurs. During the decomposition of the pyrites, there is an intermediate stage where free sulphuric acid is present before the final decomposition products, the sulphate salts, are formed. Opal is stable as far as road or other materials are concerned although, as an amorphous modification of silica with the properties of an extremely viscous liquid, it changes over geological times, first into a special crystallized modification of silica known as chalcedony, and eventually into quartz. Opal is undesirable in concrete aggregate especially when high-alkali cement is used.

There are thus two principal groups of rock-forming minerals, distinguished by (i) their prominent occurrence in different types of rock and (ii) the differences in their mode of weathering which must be considered when assessing the durability and quality of natural road building materials. The minerals which are confined to the crystalline rocks are those which may decompose if the environmental conditions are suitable. The minerals which predominate in sedimentary rocks hardly ever decompose, and disintegration, the mere physical break-down, is the dominant change which such rocks experience. A few minor exceptions to this rule will be dealt with in the appropriate places.

Quartz is the only rock-forming mineral which is stable in any environment, at least as far as its technical properties are concerned. This makes quartz one of the most widespread minerals on the surface of the earth, although it is limited to a few igneous rocks in which it is seldom the most frequent mineral. Note that in granite only about 30 per cent of the minerals are quartz but about 60 per cent are feldspar; its resistance to decomposition, however, has caused its accumulation as an unchanged weathering residue throughout the geological eras.

The rock-forming minerals occur in rocks in varying combinations and this is the basis of the classification and denomination of rocks. Generally, most igneous rocks are characterized by two or three rock-forming minerals, the metamorphic

TABLE 6

Characteristic combinations of rock-forming minerals in rocks

MAJOR ROCK GROUP	SUBGROUP	CHARACTERISTIC MINERALS	TYPICAL ROCK
Igneous	Acid	Quartz, orthoclase, mica or amphibole	Granite
	Intermediate	Orthoclase, amphibole Plagioclase, amphibole	Syenite Diorite
	Basic	Plagioclase, pyroxene	Norite, dolerite
	Ultra-basic	Pyroxene, olivine	Pyroxenite, Peridotite
Sedimentary	Clastic	Quartz	Sandstone
		Quartz, orthoclase	Arkose
		Quartz and incidental others	Conglomerate
		Clay minerals, some quartz	Shale
		Quartz, clay minerals and incidental others	Tillite, greywacke, volcanic ejecta
	Chemical precipitates	Calcite	Limestone
		Dolomite	Dolomite
		Opal and/or chalcedony	Chert
		Various salts	Gypsum and other salt deposits
	Organic	No minerals	Coal, oil
Metamorphic	Subdivisions are complicated and of little relevance to the engineering properties of the rocks	Quartz, orthoclase (occasionally mica)	Gneiss
		Quartz, muscovite (occasionally biotite)	Mica schist
		Amphibole	Amphibolite
		Quartz	Quartzite
		Calcite	Marble
		Amorphous silica or quartz and various others	Hornfels

rocks by one or two, and the sedimentary rocks by only one. In slightly more detail, this combination is shown in Table 6.

CHEMICAL ROCK ANALYSES

The only information available on a rock may be its chemical analysis. Although they reveal much to the petrologist or geochemist, such analyses are only of limited relevance to the engineering properties of a material. It must be realized that two or more rocks may be composed of a very similar combination of chemical elements which, due to differences in the conditions of formation of these rocks, have formed texturally different rocks whose mineral assemblage may also differ to some extent.

Chemical rock analyses are recorded as the percentage mass of the oxides of about a dozen chemical elements. A good chemical analysis should give a sum total of these percentages of between 99,5 per cent and 100,5 per cent. The oxides normally recorded in such an analysis are:

SiO_2, TiO_2, Al_2O_3, Fe_2O_3, FeO, MgO, Na_2O, K_2O, P_2O_5,
H_2O^+, (chemically combined water, indicated by the plus sign),
H_2O^- (free water, indicated by the minus sign), CO_2, and, in addition, occasionally MnO, SO_3 and a few others.

Since the percentage mass is recorded, such an analysis does not give a direct account of the quantity, i.e. the number of oxide molecules, contained in the material. This quantity depends on the percentage mass in relation to the molecular mass of the oxides and this mass varies much between the oxides. Thus, a given percentage mass of H_2O represents nine times as many H_2O-molecules as the same percentage mass of Fe_2O_3 would represent of Fe_2O_3-molecules. There are a number of elaborate methods to calculate the ideal mineral composition of a material whose chemical analysis is known, and often the results of calculation are surprisingly close to the actual composition. These methods use the transformation from mass percentage to molecular ratios (**Cross, Iddings, Pirsson** and **Washington**, 1903; **Niggli**, 1936); they are, however, too involved to be discussed here.

The chemical analysis of any granite, quartz-porphyry, rhyolite or granite-gneiss, the latter being the metamorphic derivative of any of the foregoing rocks, looks very much the same and it is not possible to tell from such an analysis which rock is represented (**Barth**, 1959). A few hints will be given below, however, which may help to interpret a chemical rock analysis recorded in the usual way.

SiO_2

More than 80 per cent of SiO_2 indicates that quartz or opal are the principal constituents of the rock.

65 to 80 per cent of SiO_2 indicates rock or its metamorphic derivatives with quartz as one of the major constituents.

52 to 65 per cent of SiO_2 refers to rocks which may contain some quartz, but on-

ly as an accessory. If SiO_2 is more than 60 per cent, the rock will most likely contain much orthoclase and it should still be regarded as acid for road construction purposes. If SiO_2 is less than 60 per cent, however, the principal feldspar is most likely a plagioclase and the rock should be regarded as basic. Further, if SiO_2 is less than 60 per cent in association with more than 15 per cent of Al_2O_3, more than 5 per cent of H_2O^+, more Fe_2O_3 than FeO and only small quantities of the other constituents, the analysed rock is a shale, mudstone or slate. 52 per cent down to 45 per cent of SiO_2 is typical for basic rocks, and less than 45 per cent of SiO_2 for ultra-basic crystalline rocks; these rocks are devoid of quartz. In ultra-basic rocks, plagioclase is only an accessory, if present at all, and mafic minerals are the principal constituents. In the other principal rock groups, such a low silica content is found in chemical precipitates: limestone and dolomite and their metamorphic derivatives have a high percentage of CaO, MgO and especially CO_2 in association with low or no silica, and salts are marked by high SO_3, Cl and often high H_2O^+ or H_2O^-. Impure limestone may have high SiO_2.

Fe_2O_3/FeO

The two oxides are the result of the varying valences of iron; in Fe_2O_3, ferric iron, the iron is tri-valent and in FeO, ferrous iron, it is bi-valent. The distinction between these two in chemical rock analyses is most important because in mafic minerals (biotite, pyroxene, amphibole or olivine), iron occurs in the bi-valent form but, as soon as decomposition sets in, the iron changes by oxidation into the tri-valent form. Division of the percentage mass of Fe_2O_3 by 160 and that of FeO by 72 gives the molecular ratio of these two oxides. It will be found that fresh crystalline rocks contain a greater number of FeO molecules than Fe_2O_3 molecules, while sedimentary rocks as well as decomposed crystalline rocks contain a greater number of Fe_2O_3 than FeO molecules. Chemical rock analyses which state "all iron as Fe_2O_3" are therefore of limited use.

CaO and MgO

A high percentage mass of CaO and Co_2, both being in the same order and the sum of the two being more than 80 per cent, indicates limestone or marble. This also applies to calcrete, its sum total, however, is often considerably lower and admixed sand may result in a considerably high percentage mass of SiO_2. A high percentage mass of CaO and MgO, with similar individual percentages, together with a percentage mass of CO_2 which is about equal to the sum of the percentage masses of CaO and MgO, and where the sum of CaO + MgO + CO_2 is more than 80 per cent, indicates dolomite. The same as has been said above applies to calcrete and the dolomitic variety dolocrete. A high percentage mass of MgO, but less than 10 per cent mass of Al_2O_3 and very little if any CO_2 indicates an ultra-basic rock which is rich in olivine, or in serpentine if decomposing, which latter condition is indicated by a higher percentage of H_2O^+.

H_2O^+ and CO_2

These two constituents are important indicators of weathered and sedimentary rocks and the determination of them combined as 'loss on ignition' is of little value. For the same reason, H_2O should always be separated into H_2O^+ and H_2O^-. The chemically combined water H_2O^+ forms part of certain minerals either as hydroxide (OH) or as water; H_2O^- is the free water which may be present in fine pores or fissures, often in layers not thicker than one or two molecular layers, but which can be subject to variations according to the environmental conditions. The free water can be expelled by heating the rock to 105 °C. The percentage of both types of water, depending on the overall chemical composition, and the mineral composition of the rock, increases with progressing decomposition which leads to the formation of clay or carbonate minerals. Checking of these two components is, however, only revealing if the given analysis can be compared with other analyses of the same material.

TEXTURE OF ROCKS

The size of the minerals in a rock, their state of crystallization, their geometrical arrangement and the type of bond between them determines the texture of the rock, and the texture also has an influence on the suitability for, and performance of a rock in, road structures.

Coarse-grained rocks disintegrate more easily than *fine-grained ones*. This applies to natural disintegration in the course of weathering as well as to the crushing of the rock. Coarse-grained rocks, particularly those containing large feldspars, also show numerous flat faces on the crushed stone and they are thus more inclined to detach from bitumen than finer-grained ones which produce a much more irregular surface texture when crushed (see Chapter 6).

The hardness of *amorphous material* is always less than that of the relevant crystallized modification (see Chapter 6). The resistance of amorphous material to physical break-down, either disintegration or crushing, is similar to that of fine-grained rocks, i.e. such rocks resist physical break-down or impact quite well. Amorphous material composed of anything other than pure silica, however, decomposes more easily than the crystallized mineral of the same chemical composition. The effect of the amorphous material on the polishing and adhesive properties of aggregate is variable. A rock which is composed of amorphous material only, e.g. certain cherts, abrades evenly and the overall hardness of such a material in relation to the resistance of the binder determines the degree to which the road surface becomes smooth. Similarly, a rock which is composed of amorphous material only produces smooth crushing faces and its adhesive properties are weak. If a rock is composed of crystals embedded in an amorphous matrix, e.g. quartzitic sandstone, the rock is acceptable in regard to its polishing and adhesive properties because of the difference of hardness between the amorphous and crystallized components which also result in a rough surface texture after crushing.

The *geometrical arrangement of the minerals* in a rock can be random or in some regular order. These differences in the geometrical arrangement are due to the conditions under which the rock has been formed or to secondary changes, but whatever the cause, the effect of these textural differences on the road construction properties of the rock is the same. The regular arrangement of the minerals in a rock is brought about by the tendency of those minerals whose crystal axes differ greatly in length to respond to sedimentation or pressure by arranging the longer axes parallel to each other and in the case of pressure vertical to the direction of the pressure. Amongst the rock-forming minerals, mica and amphibole are particularly inclined to enter parallel arrangements, feldspar and calcite are so inclined to a lesser degree and the others are not affected. The resulting striated texture has, of course, a weakening effect on the rock, particularly in its crushing strength, and such rocks break more easily along the planes of striation than in other directions. This tendency occurs during both natural disintegration and crushing in a plant. Massive, i.e. non-striated, rocks are therefore the stronger materials: they are more resistant to weathering, to disintegration in particular; cubical chips are obtained more easily; and they are more durable, particularly as surfacing aggregate, than rocks with a striated texture which disintegrate along the striation planes under traffic even if cubical chips have been obtained by carefully setting the rate of reduction in the crushing plant (**Shergold** and **Greysmith**, 1947; **Kilian**, 1969; **Carley-Maccauly** and **Hitchon**, 1971).

The *bond between the minerals* also affects the strength of a rock. This bond mostly obeys Van der Waal's forces (molecular bonding) as it occurs between molecules, but in rocks where minerals of the same type grow together the bond is stronger along the interfaces of such minerals than between minerals deposited as sedimentary materials, e.g. quartz as sand, in a secondary cementing matrix of a very different type, e.g. clay or iron oxide. The more similar these two components are the stronger is the bond between the cementing matrix and the predominating mineral, and therefore a quartzitic sandstone whose cementing matrix is siliceous is stronger than all other sandstones. The *shape of the grains* also contributes to the overall strength of the rock and it counteracts, to a certain degree, weak bonding forces. Irregularly shaped grains or the intergrowth of different types of mineral, e.g. that of pyroxene and plagioclase which leads to the typical ophitic texture of dolerite, provide for very strong interlock. It is obvious that the strength of the bond between the minerals as well as the shape of these minerals also affects the strength of a rock and its resistance to natural disintegration, to crushing and to impact.

STRUCTURE OF ROCKS

The fabric of rocks is described in terms of their texture and structure. Although essentially similar, the difference between texture and structure is largely one of scale: the *texture*, which has been discussed above, refers to the relationship between the individual components of a rock, mostly the minerals, such as their size, type of bond between them and their mutual arrangement; *structure* refers to the fabric of rock masses and includes stratification, fracturing, faulting and folding. Natural materials for road construction are of course not rock masses but rocks or soils which have been reduced in size naturally or artifically to the degree demand-

ed by the particular purpose or use. Texture therefore has a greater influence on their suitability as road construction materials than structure, although the latter cannot be overlooked entirely.

Stratification

The stratification of rocks is usually linked with sedimentary rocks and the way in which sedimentary material has been deposited. This is true in most cases but magma intruding between sedimentary layers, as is the case with many of the dolerite sills in the Karoo System, may also become an important part of a stratigraphic sequence.

Nomenclature

There is a certain ambiguity about the use of the words 'stratum', 'bed' and 'layer'. All three mean essentially the same thing and in the sequence above they are given in decreasing order of magnitude. Thus, the Etages of the Pretoria Series for instance generally consist of two *strata*: sandstone and shale. Within each stratum, thick *beds* of either sandstone or shale can easily be distinguished. Such beds differ from each other in having minor variations within what is generally called sandstone or shale, for example varying degrees of silicification of the sandstone, or differences in the contents of sand or organic material in the shales. The beds can then be subdivided into *layers*, which are those units which have been deposited during an uninterrupted, mostly short, period of sedimentation during which the physical conditions of deposition also did not change. Such layers are usually thin, their period of formation often being confined to a year or even only a rainy season. They are confined by a sedimentation hiatus, e.g. a dry season, which marks a passing interruption of deposition. Occasionally, however, this subdivision into stratum, bed and layer is neglected and 'stratum' is used for any of the above units:

Particularly in very fine-grained rocks, like siltstone or shale, the layers may be less than 1 mm thick and when such a rock is compressed by the increasing thickness of the overburden, the thickness of such layers is further reduced. Under such conditions, there is hardly any difference between structure and texture. The bond between the individual components of the rock is stronger within each layer than the bond between layers and, consequently, such a rock parts more easily along the planes of layering or stratification. If such a rock weathers, therefore, it disintegrates into flat pieces, the thickness of which depends on the thickness of its layers. In crushing, cubical chips may be produced from such rocks by adjusting the reduction rate properly (**Shergold** and **Greysmith**, 1947) but such crushed rock will only be of limited durability and will part along the bedding planes more easily than during natural weathering because the bond between the layers has been weakened further during the crushing process.

Certain non-sedimentary processes can lead to structures which resemble in appearance, and in their effect on the rock if used as a road building material, the stratification of sedimentary rocks. In a cooling magma which is still in motion while solidification has already partly set in, *flow structures* develop which make the rock look as if it were stratified. This occurs mostly when the magma flows on the surface, but magma which forces its way between sedimentary layers may also possess flow structures, particularly in the sill phases next to the overlying or underlying rock where cooling was fastest. The apparent layering of such a lava or magma is brought about by minerals which have already formed and which are floating in the still mobile mass. They are eventually concentrated in preferential planes within the rock. Another cause of this apparent layering is the gradual solidification of the magma in thin sheets. The resulting structure of such an igneous rock is occasionally strikingly similar to that of sedimentary rocks although certain typical differences can mostly be detected: signs of turbulence can be recognized mainly in the form of whirls, often with a mineral at the centre, and in the waviness of the 'layers'.

The road building properties of rocks with flow structure are of course basically those of igneous rocks. The bond between the 'layers' of such rocks is considerably stronger than that between sedimentary layers although it is weaker than that within the flow 'layers'. Weathering therefore proceeds a bit faster along these planes and, when such rocks are crushed, these planes act as predetermined planes of weakness.

Pseudo-stratification is very prominent in many metamorphic rocks, particularly in those in whose formation pressures played a major role. Under such conditions, the rocks are mobilized to a certain extent and their minerals change their position in such a way that their longer axes are arranged at right angles to the direction of pressure. Minerals whose axes differ greatly in length are particularly sensitive to directed pressure. Among such minerals are all micas, feldspars and amphiboles, to name a few of the rock-forming types. The result is a rock with pronounced foliation and an apparently layered structure, such as mica schist, which is again inclined to part more easily along these structural planes either during weathering or when crushed.

Deformation of rocks

Folding

Rock masses, if subjected to pressure, can react like plastic or brittle bodies depending on the rate of change of the pressure and the degree of consolidation of the rock. As long as the rate of change of pressure is such that the rock can react plastically, the rock is bent or folded. In this process, the components of the rock are subject to a certain degree of mutual movement and displacement but no shear failures occur. This leads often to the thinning of the strata, beds or layers at the rather flat jaws of a fold and their thickening at the creases, i.e. at the synclines or anticlines (*Plate 15*). The effect of bending or folding on the general engineering properties is the same as that which has been described above in connection with sedimentation, flow structures and metamorphism.

Fracturing

Highly consolidated rocks, or rocks which are subject to sudden changes of pressure or some form of natural impact, react like a rigid, brittle body. The rock is

fractured or faulted. In geology, numerous forms of fracture in rocks are known, e.g. cleavage, joints, etc., and they are important indicators of the history of rock masses and the forces transforming them. They will not be discussed here, however, and the general statements which follow will be confined to the effect of these forces on the road building properties of the rocks involved.

The result of forces which lead to fracturing may be more or less plain *cracks*, filled by air or water, if no movement of the lateral rock masses occurred. There may also be plain, often polished, slicken-sided faces in the case of minor movements of the two sides of the crack. Major relative movements, either vertical or horizontal, of the rock masses on either side of a weak zone are known as *faults* and result in intense fracturing and crumbling within the zone of faulting. Such zones of broken rock are known as *breccia*.

It is obvious that any type of fracture in a rock must affect the road building properties of this rock in some way. The joints which are found in almost all rock masses and whose causes are not yet fully understood, need not necessarily be a disadvantage because a system of rather widely spaced joints is of assistance in quarrying and the blocks, which are thus produced quite easily, can consist of entirely fresh rock which may be crushed for use as a surfacing or base aggregate.

If the fractures allow for the access of water, however, oxidation or decomposition may occur at their faces. A few millimetres of oxidized material do not seriously affect the road building properties of a rock although its use as an aggregate in bituminous surfacings is not particularly recommended (see Chapter 6); if such layers of decomposition increase in thickness, however, the quality of the rock is gradually reduced so that it is suitable only for bases.

Decomposition may reach great depths (several hundred metres occasionally) when water can circulate easily as is mostly the case in wide cracks or along the brecciated zones of faults. Quarrying in such rocks and producing from them material of uniform quality is very difficult, if not impossible (*Plate 16*). If a mixed material is all which can be obtained, the overall road building quality should be assessed from the weakest material in the mixture.

Fracture fillings

Fractures and breccias can be recemented by the clay which often develops during brecciation and which may change to a rocky substance like shale diagenetically, or by crystallization of some other mineral matter, mostly quartz or calcite but occasionally also feldspar. In this way a fairly solid rock can be formed again. The suitability for road construction of such a recemented rock depends very much on the type of cementing material and the strength of the bond between this material and the rock. Such filled fractures always remain predetermined weaker zones in the rock which will be inclined to break more easily along such structural features, usually parting by preference along one of the two faces.

The strongest filler of cracks is *quartz* and the quartz veins which are so formed are often stronger than the surrounding rock. The intergrowth between this quartz and the surrounding rock is strongest in quartzite and quartzitic sandstone which may turn to practically solid rock and become perfect road building materials even without predetermined zones of weakness along these fillings.

Feldspars as fillers of cracks are slightly weaker than quartz and, since feldspar crystals possess well-defined planes of cleavage, the filling mineral itself also breaks slightly more easily than quartz. Nevertheless, if feldspar fills cracks, particularly in feldspathic rocks like granite, the bond is still sufficiently strong to restore the road building quality of such a fractured rock.

Rocks whose cracks are filled with *calcite* always retain a predetermined zone of weakness. Such rocks are inclined to break along their calcite veins even when struck only with a hammer. Whether the filling material is actually calcite can, of course, be checked with a needle or a pocket knife, both of which will clearly scratch calcite while feldspar would remain unscratched, or by applying a drop of diluted hydrochloric acid which indicates calcite by the well-known 'boiling' reaction. Calcite also cleaves along definite planes and it is dissolved slowly if exposed to water. A rock whose cracks are filled with calcite therefore does not gain much in strength. This must be considered when selecting such rocks for use as road construction materials, particularly if crushing will be part of their processing.

Clay fillers in cracks have of course no beneficial effect on the road building properties of a fractured rock.

CHAPTER 6
GENERAL REQUIREMENTS FOR THE USE OF ROCK AND SOIL IN ROAD CONSTRUCTION

GENERAL

Fluviatile soil deposits are scarce in Southern Africa and wind-blown sand is the only geologically young, unconsolidated material which covers large parts of the country. Wind-blown sands are, however, poorly graded and the shape of their grains is generally rounded. Both properties do not render this sand particularly suitable for road construction and it has mainly been used only for selected sub-grade and occasionally in gap-graded surfacing mixtures. The scarcity of resources of acceptably graded, sufficiently angular, unconsolidated material has forced the South African road engineers to turn their attention to rock, fresh and in all stages of weathering.

SURFACING

The two types of surfacing aggregate which are mostly obtained from crushing, stone and sand, are those natural road building materials which have to satisfy the highest quality requirements in regard to the freshness of the rock, its crushing strength, resistance to abrasion, shape and mineral composition. Especially those aggregates which are used in the top layer of a bitumen surfaced road must be completely fresh and sufficiently strong not to break excessively during rolling and under traffic. Depending on the proposed use, the minimum crushing strength of the stone, as determined in the 10 per cent Fines Aggregate Crushing Test (10 per cent FACT) (**British Standards Institution**, 1975a; **National Institute for Transport and Road Research**, 1979), should be between 160 and 210 kN for the dry material, provided that the wet test does not show less than 75 per cent of the strength obtained on the dry material (**South African Bureau of Standards**, 1976). By changing the rate of binder application slightly, however, limited allowance can be made for slightly weaker or more porous stone. These adjustments will ensure the correct quantity of bitumen in relation to the pore volume which can be expected after rolling (**National Institute for Road Research**, 1971).

The actual strength requirements vary among the different road authorities and much depends on the traffic the road is expected to carry. In the Republic of South Africa, for instance, a strength of at least 210 kN (ACV: 21 per cent) is required for a surface treatment, in contrast to Rhodesia where 150 kN (ACV: 26 per cent) is acceptable. Similarly, the strength requirements for the coarse aggregate in a continuously graded asphalt surfacing are at least 160 kN (ACV: 25 per cent) in the Republic of South Africa and only 80 kN (ACV about 32 per cent) in Rhodesia.

The strength requirements should also vary with the type of rock, and the specification of just one minimum value for all rocks is not the entirely correct ap-

proach. Since there is no uniform entity 'rock', there is also no uniform minimum strength which can be applied equally to all rocks. This important matter is dealt with in more detail in Chapter 8.

POLISHING

Stone exposed to abrasion by traffic will attain a smooth surface in time, i.e. it will polish (*Plate 17*). A minimum polished stone value of 45, determined in accordance with BS812:1975 (**British Standards Institution**, 1975a), should apply to aggregate used on ordinary carriageways and this value should be higher on parts of roads which are liable to more intense abrasion, e.g. on sharp rises, approaches to intersections and on more heavily trafficked roads.

The degree of polishing of a stone depends on its mineral composition and the hardness of the minerals can be taken as a measure of their potential resistance to abrasion. The occurrence of minerals of similar or different hardness in a rock therefore determines the potential degree of polishing of the whole stone and the rate at which it proceeds.

Only a few of the many thousands of minerals occur sufficiently frequently in rocks to influence polishing. These minerals are tabulated below in decreasing order of hardness:

TABLE 7

Hardness of rock-forming minerals

Mineral	Hardness (Mohs' scale)
Quartz	7
Olivine	6,5-7
Epidote	6-7
Pyroxenes	6-6,5
Feldspars	6
Opal	5,5-6,5
Hornblende	5,5-6
Dolomite	3,5
Calcite	3
Micas	2,5-3
Chlorite	2-2,5
Clay minerals	1-2

The table shows the importance of quartz. Only two of the more important rock-forming minerals may be as hard as quartz, but while epidote and quartz may occur together in certain metamorphic rocks, olivine is not found in a rock which also contains quartz. Rocks which contain both epidote and quartz are not very common and in Southern Africa they have so far only been used occasionally and with success in the Kariba area in Rhodesia. All other minerals are markedly less resistant to abrasion and therefore rocks containing quartz in association with other minerals are always less liable to polishing than quartz-free rocks.

It is obvious from the above that certain rocks, e.g. granite (quartz + feldspar + mica or other dark minerals), tillite (quartz + opal + clay or other soft minerals), quartzitic sandstone (quartz + opal; often incorrectly called 'quartzite') and calcrete, provided its host material is sand, give higher values in the polishing test than others, e.g. dolerite (feldspar + pyroxene). Dolomite gives very low values because of the uniformly low hardness of its mineral constituents. All rocks which are composed of only one mineral polish in time, even if this mineral is quartz, as is the case in quartzite. Most of these monomineralic rocks, e.g. dolomite, are a serious hazard to the skid resistance of a road and this would apply also to the hard quartzite if the binder did not abrade so much faster than the stone, thus ensuring that the road retains a skid-resistant surface.

A slightly weathered rock which consists of minerals of almost the same hardness, e.g. dolerite or norite, often polishes less than a completely fresh rock of the same type. This can happen under some special environmental conditions when one type of mineral starts weathering earlier and faster than the others. If such a process is in a very early stage, a varying degree of resistance to abrasion develops in the rock and the fresh and more resistant minerals make the rock retain a rough surface. Such rocks will not be used frequently, however, since they will seldom meet the strength requirements because the minerals, which are already slightly weathered, affect the crushing strength adversely.

ADHESION

Adhesion is the property that certain substances have of 'sticking together' and this may be achieved by a combination of a number of mechanisms. These mechanisms cause the bonding of particles either by strong chemical or electrical forces, or by the forces associated with absorption or diffusion (which are several orders of magnitude weaker) with or without mechanical interlock. These weak bonds are intermolecular and controlled by Van der Waal's forces.

In the adhesion of a bitumous binder to stone, electrical and mechanical forces as well as those of adsorption and diffusion play a role though the importance of the latter is a bit uncertain. There is, however, no chemical reaction between binder and stone, i.e. no new chemical compounds are formed, and chemical adhesion therefore does not occur in this case. Nevertheless, the thought given to whether to use either anionic or cationic emulsions, with 'basic' or 'acid' rocks respectively, shows some thinking in terms of the chemical nature of the adhesion between binder and stone. This is not the place to elaborate on the principles of adhesion and reference should rather be made to comprehensive books such as the 'Textbook of Physical Chemistry' by **Glasstone** (1956).

The role of the stone in its interaction with a bituminous binder can apparently be conceived of in three ways: the chemical concept, the mechanical concept and the interface energy concept. These concepts will be discussed below with reference to the aggregate and it may be noticed that a number of different mechanisms of adhesion is contained in each of them.

The chemical concept

In the chemical concept the aggregate is described as 'hydrophilic', i.e. water-loving or 'hydrophobic', i.e. water-hating, and either acid or basic, the latter terms referring to the so-called acid or basic rocks. These latter two terms are, however, based on the assumption by petrologists of the 19th century that the silica (SiO_2) in igneous rocks is the radical of silicic acid. Igneous rocks which contain sufficient silica to satisfy the silica demand of the other constituents of a magma and to produce free quartz or a very large quantity of orthoclase (feldspar), as in syenite, have been called 'acid' in contrast to those without free quartz which are referred to as 'basic'. Neither of these terms must be understood in the chemical sense, however, and they are being abandoned nowadays in favour of terms for the grades of saturation of a magma with silica.

The chemical concept states that a rock (or aggregate) is 'acid' if the 'acidic' compounds, SiO_2 and CO_2, comprise more than 50 per cent of it. This limit of 50 per cent could refer to:

a) the percentage mass as determined in a chemical analysis, or
b) the percentage of the number of relevant molecules.

a) That the limit of 50 per cent of the sum of SiO_2 and CO_2 refers to percentage mass would be the normal way of thinking. SiO_2 occurs in rocks either as a component of the silicate minerals or as quartz, while CO_2 occurs only as part of the carbonate minerals, e.g. calcite; $CaO + CO_2 = CaCO_3$.

Considering igneous rocks first, attention need only be drawn to SiO_2 because, except in very special cases as, for example, in carbonatites, a type of igneous rock which is composed mainly of primary calcite, CO_2 plays no role in these rocks. Many basic rocks contain more than 50 per cent by mass of silica and South African dolerites, norites, basalts and other basic rocks used in road surfacings would very often classify as 'acid' according to the percentage by mass of silica. The large majority of sedimentary rocks and most metamorphic rocks also contain more than 50 per cent by mass of silica to which carbon dioxide (CO_2) may be added in many cases. Only carbonate rocks, i.e. limestone, dolomite and their metamorphic derivative, marble, would be 'basic' in most cases, because only a considerable quantity of quartz which nearly always occurs in these rocks in the form of sand grains, will bring the sum of carbon dioxide and silica above 50 per cent by mass.

In Table 8, 52 different rocks are arranged according to the average percentage by mass of their silica and carbonate contents. This Table shows a definite concentration of those rocks which, according to TRH 7 (**National Institute for Road Research**, 1972), would not make suitable aggregate when anionic bitumen emulsions are used. Most of these rocks occur in the top portion of the arrangement based on the percentage by mass of the sum of silica and carbon dioxide. This must be expected, of course, in an arrangement which uses this sum value as its principle. It is obvious, however, that many 'basic' rocks which can satisfactorily be used with an anionic emulsion contain more than 50 per cent by mass of silica and carbon dioxide. The Table shows further that, if a limit were to be set, it should

TABLE 8

Contents of $SiO_2 + CO_2$ in rocks (in per cent mass)

Rock name	Near equivalent to rocks in Table 1, page 7 of TRH7	Suitability for use as aggregate with anionic bitumen emulsions doubtful: ?	Percentage by mass of sum of $SiO_2 + CO_2$
1. Quartz	Quartzite	?	100,00
2. Quartzitic sandstone	Sandstone	?	94,60
3. Vein quartz	Quartzite	?	94,26
4. Sparagmite	Tillite	variable	80,90
5. Arkose	Sandstone	?	75,90
6. Rhyolite	Rhyolite	?	72,80
7. Granite	Granite	?	70,18
8. Greywacke	Tillite	variable	65,80
9. Dacite	Felsite	?	65,68
10. Granodiorite	Granite	?	65,01
11. Quartz latite	Andesite		62,43
12. Quartz diorite	Andesite		61,59
13. Shale	Hornfels		60,70
14. Trachyte	Syenite	?	60,68
15. Syenite	Syenite	?	60,19
16. Andesite	Andesite		59,59
17. Trachy-andesite	Andesite		57,84
18. Phonolite			57,45
19. Diorite			56,77
20. Monzonite	Syenite	?	56,12
21. Quartz basalt	Basalt		55,46
22. Nepheline syenite			54,63
23. Quartz gabbro	Norite		54,39
24. Spessartite			53,52
25. Vogesite			52,62
26. Magnesite	Dolomite		52,38
27. Kersantite			50,79
28. Anorthosite	Dolerite		50,40
29. Amphibolite	Amphibolite		49,99
30. Minette			49,45
31. Trachy-dolerite	Dolerite		49,20
32. Tephrite	Basalt		49,14
33. Basalt	Basalt		49,06

Rock name	Near equivalent to rocks in Table 1, page 7 of TRH 7	Suitability for use as aggregate with anionic bitumen emulsions doubtful: ?	Percentage by mass of sum of $SiO_2 + CO_2$
34. Shonkinite	Syenite	?	48,66
35. Essexite	Norite		48,64
36. Gabbro	Norite		48,24
37. Dolomite (mineral)	Dolomite (rock)		47,82
38. Limestone	Marble		46,77
39. Urtite			45,61
40. Theralite	Norite		45,61
41. Monchiquite			45,17
42. Basanite	Basalt		44,64
43. Calcite	Marble		44,00
44. Ijolite			42,81
45. Nephelinite	Dolerite		41,17
46. Camptonite			40,70
47. Dunite			40,49
48. Nepheline basalt	Basalt		39,87
49. Alnöite			32,31
50. Haematite			0,00
51. Magnetite			0,00
52. Rock salt			0,00

rather be placed at 60 per cent, neglecting the few syenite-type rocks which contain less than 60 per cent and even less than 50 per cent of silica and carbon dioxide.

If SiO_2 were considered in the form of quartz, only rocks like quartzite and sandstone would be regarded as 'acid' while the quartz contents of most other rocks would remain below 50 per cent. It is very unlikely, however, that this has ever been considered as such in the chemical concept because CO_2 does not form minerals of its own like SiO_2 and it would be of no use then to work with the sum of $SiO_2 + CO_2$

b) The percentage of mass as recorded in chemical analyses is of little use for obtaining information on the properties of a rock.

The mass of the constituents found by such an analysis varies considerably and the percentage mass is therefore no direct indication of the quantity of the different molecules in the rock. Twelve or thirteen oxides are usually determined in rock analyses and their very different molecular masses are given in Table 9.

TABLE 9

Molecular masses of the constituents of a rock as determined by chemical analysis

Oxide	Molecular mass
SiO_2	60
Al_2O_3	102
Fe_2O_3	160
FeO	72
MnO	71
MgO	40
CaO	56
Na_2O	62
K_2O	94
TiO_2	80
P_2O_5	142
H_2O	18
CO_2	44

In very detailed analyses a few more oxides and, in addition, sulphides and other components, are also determined but those above are sufficient for most purposes of interpretation. When the percentage mass of each oxide is divided by its molecular mass, a ratio called the 'molecular fraction' or 'molecular proportion' (**Cross** *et al*, 1903, 1912; **Washington**, 1917; **Holmes**, 1930) is obtained which can be used as an expression for the number of molecules of the constituent concerned and thereby the percentage of the relevant molecules in the rock can be determined. Table 10 lists the same rocks as in Table 8 against the percentage of the sum of the number of silica and carbon dioxide molecules.

Again the concentration of the 'acid' rocks which are supposed to perform unsatisfactorily with anionic emulsions appears at the top of the Table, but again there is also a wide overlap of 'acid' and 'basic' rocks. Although the percentage of the sum of SiO_2 and CO_2 molecules is higher than their percentage mass, due to the relatively low mass of these oxides (Table 9), a limit set at 60 per cent would again have been the better choice.

Neither approach is conclusive to such an extent that reliable predictions could be made in regard to whether to use anionic or cationic bitumen emulsions. The negative charge in those rocks which are commonly used as surfacing aggregate is carried by the oxygen ions and, since the number of oxygen atoms in rocks (see Table 11) varies between 52 per cent in alnöite, an extremely ultra-basic dyke rock, and 66,7 per cent in pure quartz, this would make rocks rather strongly negatively charged entities if all the atoms in them were ionized.

Ionization of atoms occurs only when they enter into chemical compounds by ionic bonding, i.e. where one atom loses an electron, or electrons, from its outer shell and another atom adds this or these electrons to its outer shell. The degree of ionic bonding depends on the electronegative difference between the two atoms and there are no two elements on earth for which this difference is such that they only enter ionic bonding. This means that part of the chemical bond between any

two atoms is always covalent, i.e. electrons on the outer shells are only shared but not lost or gained, in which case no ionization occurs.

In minerals and rocks, ionic and covalent bonds are about equally distributed and, considering the number of charged i.e. ionized, atoms in a rock, just more than 30 per cent of the atoms in a rock are anions, all being oxygen ions, and just more than 20 per cent are cations.

These values are very much the same for all types of rock. In general, therefore, all rocks have a weak negative charge and there is no justification for explaining the different adhesive properties of binders through a difference between 'acid' and 'basic' rocks.

The mechanical concept

The mechanical concept of adhesion considers only the degree of interlock between a bituminous binder and the stone. In this concept, only the geometrical shape of the stone surface, i.e. flat and smooth faces, surface irregularities, protrusions, indentations and cavities, is considered in such a way that the more irregular this surface, the better the binder will adhere to the stone. Although it appears quite feasible, this concept does not offer an explanation for the actual adhesion of the binder to the stone.

The interface energy concept

The mechanical concept used in conjunction with the interface energy concept provides, however, a means to understand part, although not all, of the role played by the stone in obtaining the desired bond with bitumen in road surfacings. Each stone surface, in fact any surface, possesses a certain amount of free energy. This energy is derived from those parts of the electron cloud around the atoms in the surface which cannot find a suitable bond inside the piece concerned. This free energy tries to attract to the stone surface whatever can be attracted and, as a consequence, each surface of a freshly crushed rock is covered virtually immediately by a thin film of water, several molecular layers thick.

This free energy also enables other materials to 'stick' to such a surface as long as these materials possess some polarity, as road binders do. The mechanism of the interaction between rock, water and bitumen will not be discussed here since there are other competent publications dealing with this (**Thelen**, 1958; **Karius** and **Dalton**, 1964). It is obvious, however, that a flat, smooth surface provides less free energy than a surface with numerous protrusions where the atoms, sitting at the edge of such protrusions, provide considerably more free energy than the atoms in a smooth surface. A rough piece of stone, therefore, provides for a much stronger bond at many places on its surface than a smooth one. It may be argued that the surface energy at the bottom of indentations is less than that on a smooth surface, for the same reason that it is greater at the edges of protrusions, and that the total amount of surface energy per unit surface area therefore remains constant. This is true, but there are not only the protrusions with their particularly high surface energy potential, but the total surface area of the stone with rough surfaces is also

TABLE 10
Percentage of molecules of SiO_2 and CO_2 in rocks

Rock name	Near equivalent to rocks in Table 1, page 7 of TRH 7	Suitability for use as aggregate with anionic bitumen emulsions doubtful: ?	Percentage of sum of molecules of SiO_2 and CO_2
1. Quartz	Quartzite	?	100,00
2. Quartzitic sandstone	Sandstone	?	94,64
3. Vein quartz	Quartzite	?	94,26
4. Sparagmite	Tillite	variable	84,41
5. Arkose	Sandstone	?	80,95
6. Rhyolite	Rhyolite	?	76,48
7. Granite	Granite	?	75,00
8. Dacite	Felsite	?	69,48
9. Granodiorite	Granite	?	69,00
10. Quartz diorite	Andesite		66,78
11. Greywacke	Tillite	variable	66,69
12. Trachyte	Syenite	?	66,47
13. Quartz latite	Andesite	?	66,10
14. Syenite	Syenite		64,58
15. Andesite	Andesite		63,45
16. Trachy-andesite	Andesite		63,05
17. Phonolite			62,09
18. Shale	Hornfels		60,69
19. Nepheline syenite			60,37
20. Monzonite	Syenite	?	60,37
21. Diorite			59,76
22. Quartz basalt	Basalt		58,37
23. Quartz gabbro	Norite		56,65
24. Anorthosite	Dolerite		55,78
25. Spessartite			55,03
26. Shonkinite	Syenite	?	54,12
27. Trachy-dolerite	Dolerite		53,30
28. Urtite			52,52
29. Limestone	Marble		52,51
30. Vogesite			52,36
31. Essexite	Norite		52,05
32. Basalt	Basalt		52,04
33. Tephrite	Basalt		51,70
34. Amphibolite	Amphibolite		51,61
35. Magnesite	Dolomite		50,00
36. Dolomite (mineral)	Dolomite (rock)		50,00
37. Calcite	Marble		50,00
38. Gabbro	Norite		49,88
39. Minette			49,43
40. Kersantite			47,91
41. Theralite	Basalt		47,65
42. Ijolite			47,13
43. Basanite	Basalt		45,81
44. Monchiquite			45,20
45. Nephelinite	Dolerite		44,17
46. Nepheline basalt	Basalt		40,40
47. Camptonite			39,49
48. Dunite			33,12
49. Alnöite			27,86
50. Haematite			0,00
51. Magnetite			0,00
52. Rock salt			0,00

larger relative to the volume of the stone than the surface area of a stone with smooth surfaces, and the resulting total surface energy increases accordingly. The effect of the increased total surface energy on the adhesion of the binder to the stone is further supported by the merely mechanical interlock which is produced by a rough surface texture.

It can be concluded from these considerations that the strength of adhesion of the road binder to the stone depends, as far as the stone is concerned, on its surface texture: the more unevenly textured the surface, the stronger the adhesion is, and the smoother the surface the weaker the adhesion. The quantity of silica (SiO_2) or carbon dioxide (CO_2), i.e. whether the rock is 'acid' or 'basic', is of little importance. All rocks are weakly negatively charged and there is no noticeable difference in regard to the total charge between the different types of rock.

The chemical composition of rocks has, however, an indirect influence on the surface texture which is obtained after crushing. If only a limited number of chemical components make up the bulk of the mass of a rock, the number of different types of mineral in such a rock is small and the individual minerals may attain a relatively large size. In most cases, silica (SiO_2) is the component in rocks

TABLE 11

Percentage of oxygen atoms and ions in rocks

Rock name	Near equivalent to rocks in Table 1, page 7 of TRH 7	Suitability for use as aggregate with anionic bitumen emulsions doubtful: ?	Percentage of sum of oxygen atoms and ions
1. Quartz	Quartzite	?	66,67
2. Quartzitic sandstone	Sandstone	?	65,88
3. Vein quartz	Quartzite	?	65,43
4. Sparagmite	Tillite	variable	64,30
5. Arkose	Sandstone	?	63,00
6. Granite	Granite	?	62,06
7. Rhyolite	Rhyolite	?	61,77
8. Granodiorite	Granite	?	61,40
9. Quartz basalt	Basalt		61,40
10. Quartz diorite	Andesite		61,19
11. Syenite	Syenite	?	61,14
12. Dacite	Felsite	?	61,03
13. Andesite	Andesite		60,79
14. Greywacke	Tillite	variable	60,66
15. Anorthosite	Dolerite		60,56
16. Quartz latite	Andesite	?	60,51
17. Monzonite	Syenite		60,32
18. Diorite			60,26
19. Trachyte	Syenite	?	60,14
20. Trachy-andesite	Andesite		60,12
21. Limestone	Marble		60,04
22. Dolomite (mineral)	Dolomite (rock)		60,00
23. Calcite	Marble		60,00
24. Magnesite	Dolomite		60,00
25. Haematite			60,00
26. Quartz gabbro	Norite		59,68
27. Spessartite			59,33
28. Shonkinite	Syenite	?	59,33
29. Essexite	Norite		59,30
30. Amphibolite	Amphibolite		59,29
31. Gabbro	Norite		59,27
32. Trachy-dolerite	Dolerite		59,25
33. Basalt	Basalt		59,23

Rock name	Near equivalent to rocks in Table 1, page 7 of TRH 7	Suitability for use as aggregate with anionic bitumen emulsions doubtful: ?	Percentage of sum of oxygen atoms and ions
34. Nepheline syenite	Basalt		59,03
35. Tephrite			58,87
36. Phonolite			58,68
37. Shale	Hornfels		58,51
38. Vogesite			58,28
39. Ijolite			58,00
40. Basanite	Basalt		57,87
41. Theralite	Basalt		57,31
42. Urtite			57,25
43. Magnetite			57,14
44. Nepheline basalt	Basalt		57,12
45. Nephelinite	Dolerite		57,09
46. Monchiquite			56,97
47. Minette			56,67
48. Kersantite			56,02
49. Camptonite			55,33
50. Dunite			55,32
51. Alnöite			52,50

(In rock salt, the negative charge is carried by the Cl-ion and the relevant percentage would be 50.)

which may be superior to such a degree that the other constituents are notably suppressed. The resulting rock is of that type which is commonly called 'acid', e.g. granite but also quartzite, and which is normally characterized by rather large minerals, particularly the orthoclase feldspar in the case of granite. On crushing, such a rock produces many smooth faces or smooth areas within such faces. If silica becomes so pronounced that it is the only or nearly the only component of the rock, as in vein quartz, the extremely strong bond between the quartz minerals causes a fracture, as developed during crushing, to pass right through the minerals and smooth surfaces are again the result. If much or most of the rock consists of amorphous silica and its bond with other minerals, particularly quartz, is sufficiently strong, e.g. in quartzite or hornfels, smooth crushing faces are again produced. Also in cases where another component is present in above average quantity, e.g. potassium in syenite, large individual minerals (orthoclase feldspar in the case of syenite) develop and crushing leads to smooth, flat faces. There is no such tendency to produce flat, smooth crushing faces in 'basic' rocks which contain largesized minerals only in exceptional cases. These differences in the type of crushing face formed in the different types of rock have probably led to the impression that

'acid' rocks cause more problems with adhesion than do 'basic' ones.

The criteria for a surfacing stone are therefore:

a) The rock must be sufficiently strong not to break or crush during rolling or under traffic. Although the minimum strength requirements are usually of the order of 160 to 210 kN in the 10 per cent FACT on the dry material, the precise specifications of the different road authorities vary considerably. Moreover, different types of rock require different strength values (see Chapter 8).

b) The rock must not abrade excessively under traffic. It should have a polished stone value of at least 45, or more if the traffic demands. Rocks which are composed of minerals of different hardness, preferably quartz in association with other softer minerals, polish least.

c) In black-top surfaces, the stone must adhere well to the binder. Rocks which possess a rough surface texture after crushing and which do not contain large minerals from which smooth cleavage faces develop, are more suitable than those which give smooth crushing faces.

The most suitable surfacing aggregate is therefore obtained from medium-grained, strong, fresh rock which contains quartz in association with other minerals or quartz in a strong cementing matrix (**Hartley**, 1974). Such materials may not always be available but, by taking additional care in the design and construction of the surfacing, slightly inferior material may well be used especially in lightly to medium-trafficked roads.

The importance of the grain size of the aggregate must also not be overlooked. The above considerations were concerned primarily with the coarse aggregate, the stone, which usually has a diameter of more than 4,75 mm. In the smaller sizes, the 'sand' of bituminous mixtures, however, the average condition of the rocks which has been discussed so far may become subsidiary to that of the individual minerals. Little difference will probably be experienced between the durability of adhesion of coarse and fine aggregate from cryptocrystalline or very dense rocks, such as limestone or quartzite, because there is no difference between the texture and the mineral composition of the various grain sizes. This does not apply to crystalline rocks, i.e. rocks whose individual minerals can be recognised with the naked eye or at least with a hand lens. The fine aggregate obtained from such rocks will eventually consist of individual minerals whose properties may be different from those of larger pieces of the whole rock. The mineralogical composition of a rock, which is not necessarily the same as its chemical composition, may therefore have a certain influence on the adhesion of bitumen to the aggregate (**Peterson** *et al*, 1974) and the differences between satisfactory and unsatisfactory adhesive properties of minerals are probably more obvious on the fine than on the coarse aggregate.

The shape of the fine aggregate in bituminous mixtures also has some influence on their performance. Well-rounded grains, e.g. wind or water-worn sand, perform less well than angular, so called 'sharp' grains. This is due to the restricted interlock between the rounded grains which makes them mobile to some extent within the mixture, and also to the weaker bond between the grains and the bitumen in a similar form as applies to the surface of coarse aggregate.

PLATE 1: The Via Appia, built by the Romans during the 3rd century B.C., parts of which can still be used today. *(A. Tomeucci.)*
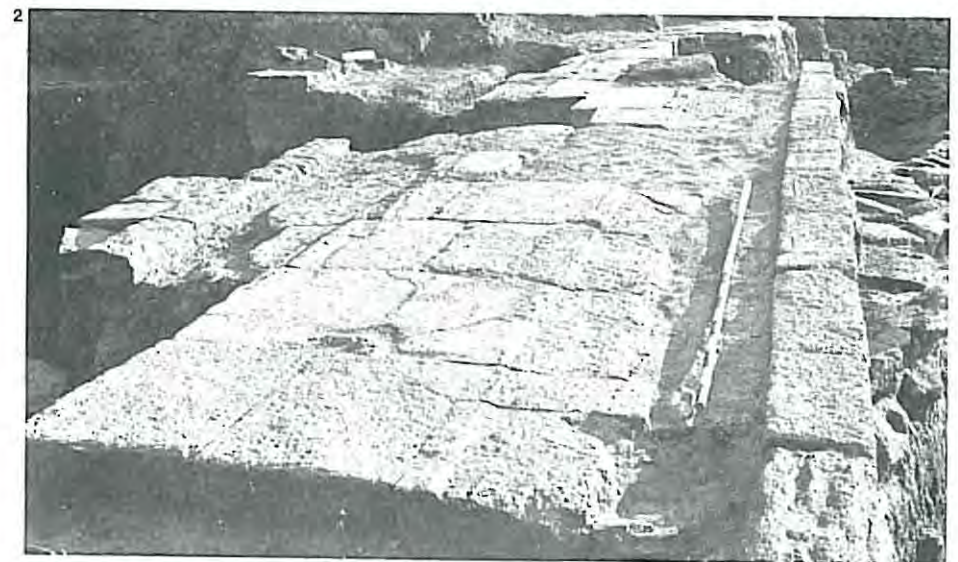
PLATE 2: Processional way of Assur (Mesopotamia) as reconstructed during the 7th century B.C. The slabs were dressed to serve as 'rails' for the processional wagons. The Aibur Shabu at Babylon was structurally similar. *(Staatliche Museen zu Berlin.)*

PLATE 3: Cobbled street in a Central European town. *(Klaus Lehnartz, Pressefoto, Berlin.)*

PLATE 4: Modern freeway in South Africa.

PLATE 5: Fresh norite covered by humic soil near Narvik, Norway.

PLATE 6: Pebble marker separating biotic soil above from soil residual from norite below. Note that the quartz veins are cut off at the pebble marker.
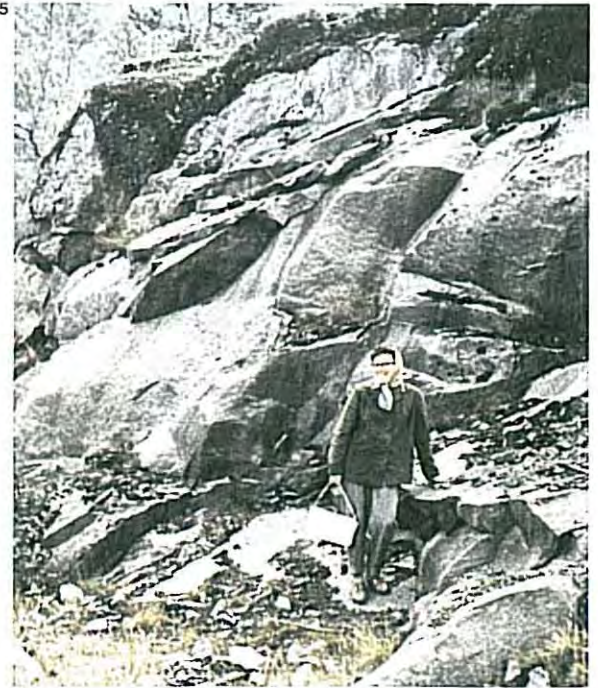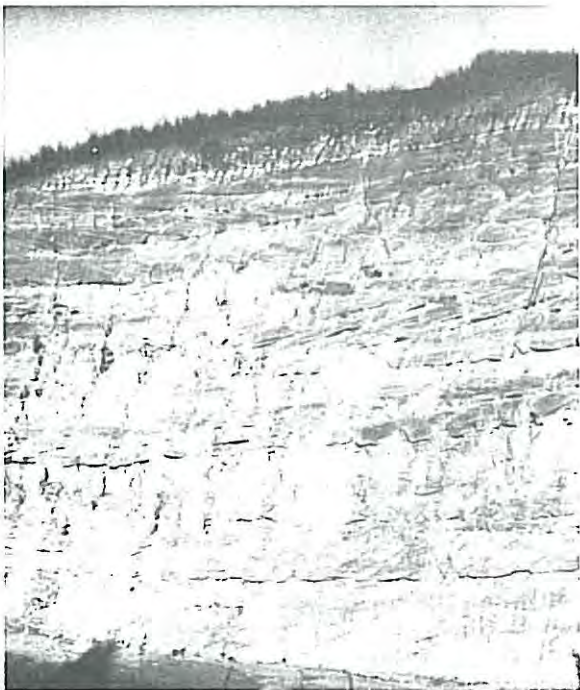
PLATE 7: Road cutting in sandstone with numerous seepage horizons.

PLATE 8: Road shoulder deformed by moles.

PLATE 9: Aerial view of borrow 'areas' south of Springbok, Cape Province, where N is about 25. *(Dept. of Transport, Pretoria.)*

PLATE 10: Borrow 'area' about 150 mm deep near Ariamsvlei, Cape Province, where N is about 40. *(Dept. of Transport, Pretoria.)*

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PLATE 11: Borrow pit. *(Dept. of Transport, Pretoria.)*

PLATE 12: Disintegrated granite south of Vioolsdrif, Namaqualand where N is about 50. *(Dept. of Transport, Pretoria.)*

PLATE 13: Lime nodules concentrated in the lower half of black clay overlying highly decomposed norite. The N-value of the site is between 2 and 3.
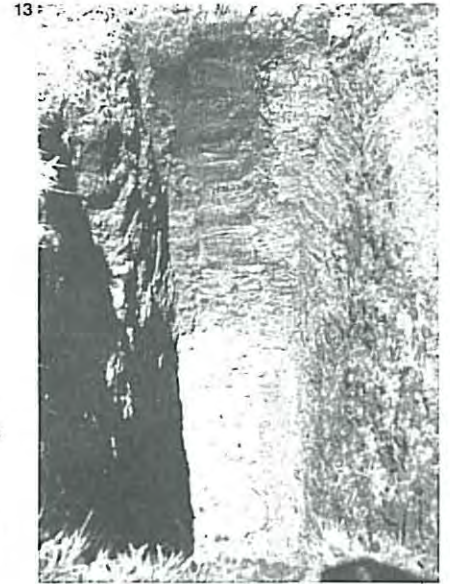
PLATE 14: Highly decomposed dolerite covered by residual black clay. A few dolerite boulders lie on the floor of the pit. Near Dealsville, Orange Free State, where N is about 3.

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PLATE 15: A fold. Note the thickening of the layers at the creases. Ugab River, Damaraland, South West Africa. *(F. D. I. Hodgson.)*

PLATE 16: Differential and deep decomposition along cracks in dolerite. Such irregular decomposition creates problems with the recovery of suitable material. Near contour of N = 5.

PLATE 17: Polished road surface. *(Dept. of Transport, Pretoria.)*

PLATE 18: Dust on calcrete road.

PLATE 19: Cracked and deformed surfacing due to failure of base material.

PLATE 20: Photomicrograph of fresh dolerite.
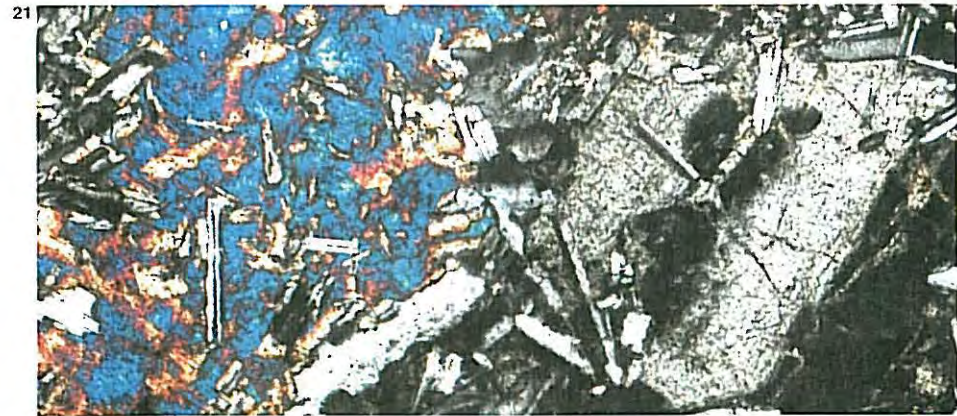
PLATE 21: Photomicrograph of decomposed dolerite. Outlines of minerals well defined in most cases but many minerals more or less severely cracked.

PLATE 22: Photomicrograph of highly decomposed dolerite. Outlines of primary minerals largely ill-defined and remaining primary minerals all severely cracked; dark and 'washed' areas are secondary minerals.

PLATE 23: Photomicrograph of soil residual from dolerite. Remaining primary minerals in sharp contrast to secondary minerals. Severely cracked primary minerals in the state of final decomposition.

PLATE 24: Tillite quarry with 'second brown' tillite right on top and in two major cracks dipping to the right-hand side, 'first brown' tillite to about half-way down the profile and 'blue' tillite below.
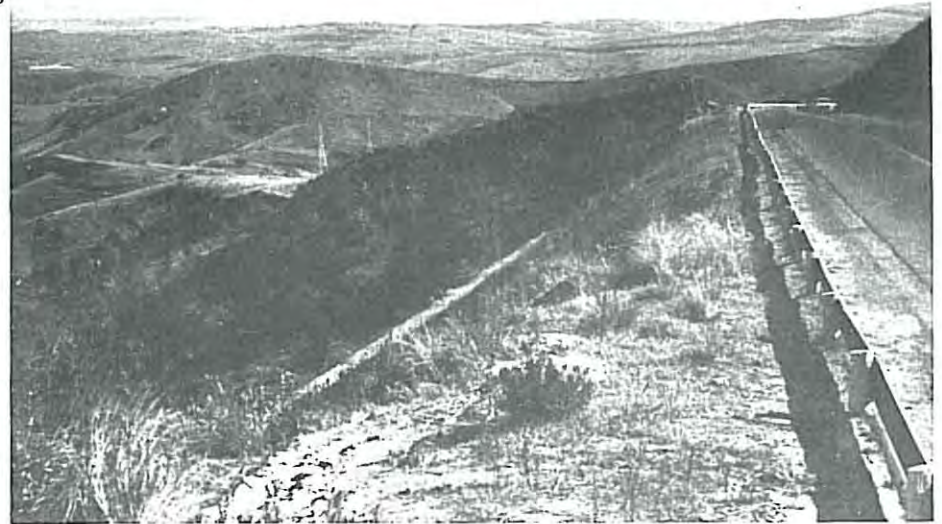
PLATE 25: Calcrete hardpan overlying nodular calcrete near Rundu, South West Africa, where N is between 4 and 5.

PLATE 26: Calcrete road with 'blad'.

PLATE 27: Boulder calcrete.

PLATE 28: Road cut in upper portion of talus slope. Any disturbance in the steeper toe portion of the slope, e.g. by another cut, may lead to serious failure.

PLATE 29: Collapse of road caused by sinkhole. (Geological Survey, Pretoria.)

PLATE 30: Sinkhole in borrow pit next to road.

PLATE 31: Large crack on the top of slope of road cut. Ingress of water may cause a rock slide.

PLATE 32: 'Cat steps' indicating soil movement down-slope. Note slump area behind 'cat step' slope. (A. B. A. Brink.)

PLATE 33: Dry, severely cracked montmorillonite clay.

PLATE 34: Loss of shape due to insufficient bearing capacity of the subgrade.

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PLATE 35: Salt blisters. Note the white salt filling of opened blisters.
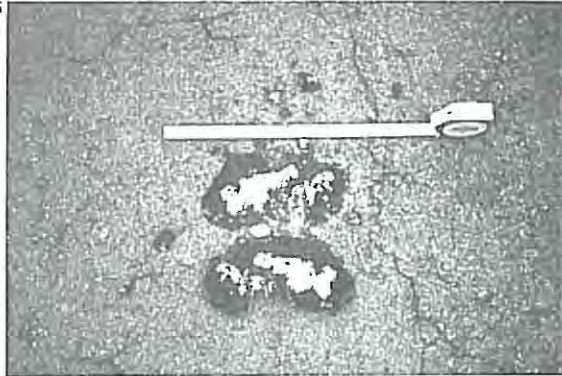
PLATE 36: Photomicrograph of 'sunburner' basalt. The black areas are unaffected rock. Note the numerous hair cracks, showing in white, radiating from affected areas. (*H. Vogler, Krefeld.*)

PLATE 37: Damage to road caused by collapsing soil: note the angle of the truck.
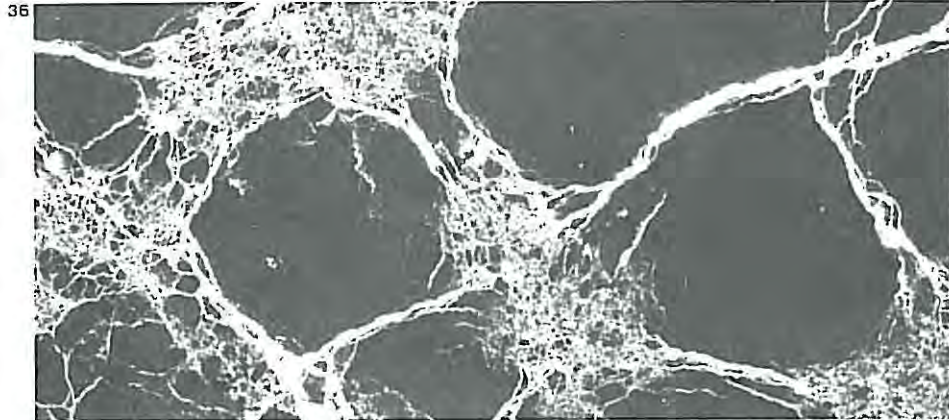
The purpose of the layers below the surfacing is to provide successive working platforms as construction work progresses, and these platforms must be designed so that overlying or underlying layers are not overstressed under the load of the traffic. The level surfaces are obtained by properly shaping whatever material is used and, except for the grain-size distribution, the petrological type of the material is of little importance. The even spreading of the load, however, requires materials with adequate physical properties and of sufficient durability to retain these properties during the whole design life of the road.

This book does not deal with pavement design and a detailed discussion of the differences between the layers of a pavement is not its purpose. The quality requirements of the natural materials for these layers can be discussed in sufficient detail by considering them to consist of base, subbase, (selected) subgrade, the improved subgrade or fill and the natural soil, frequently called the roadbed. Depending on the type of road, the pavement may consist of all these layers or only some of them and, with reference to the required quality of the materials, they can be arranged as follows:

	Asphalt pavement	Concrete road	Gravel road
Materials quality increasing →	Bituminous surfacing		
	Base	Concrete pavement	
	Subbase	Subbase	Gravel wearing course
	Subgrade (selected)	Subgrade (selected)	Subgrade (selected)
	Subgrade or fill	Subgrade or fill	Subgrade or fill
	Roadbed	Roadbed	Roadbed

It is obvious that the most complex structure is that of the asphalt pavement (Figure 10) and the simplest one that of the gravel road. The above arrangement refers to the quality requirements for the materials in the structural layers of a pavement and it is set out so that the pavement layers shown in the same line, i.e. next to each other, require materials of similar quality.

The previously clear distinction between the structural layers and their material requirements has become somewhat indistinct recently due to a greater variety in road design following the increase of traffic (**National Institute for Transport and Road Research**, 1978c (TRH4)). Consequently there are types of material which may be suitable for the base of certain roads but only for the subbase of others (National Institute for Transport and Road Research, 1980 (TRH14)). This has resulted in a considerable variability in the use of untreated and differently treated materials whose individual descriptions would demand much unnecessary repetition. The above conventional layers have therefore been retained in the following discussion.
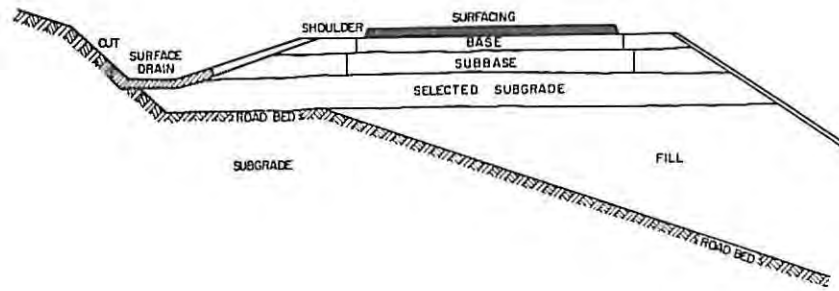
Figure 10: *Structure of a bituminous pavement*

BASE

The design of the base must be such that the stresses exerted by the traffic are so spread over the underlying layers as not to overstress either the surfacing or the subbase. This requires the use of strong materials which must also be durable so that the design test results are not invalidated by early excessive alteration in the materials. This can only be achieved by most careful selection and it is often most important, particularly in parts of Southern Africa where N is less than 5, not to economise on haulage; quality must be maintained.

Up to the mid-sixties, base material was virtually exclusively obtained from sources of weathered rock. Crushed stone bases were the exception, being built mostly on the Reef, occasionally in the Cape Province and in municipal areas. The introduction of freeways about a decade ago changed the situation considerably through the increased use of crushed rock for base construction, particularly in the Republic of South Africa.

Crushed stone bases

These bases were previously known as crusher-run bases.

As the name implies, crushed fresh rock is used for these bases. This means that the quality requirements of the material, the fresh rock, are similar to those of the surfacing aggregate, but some relaxations are permissible with regard to strength, freshness and cleanliness. The required minimum strength is mostly 110 kN (ACV 29%) in the 10% FACT on dry material, provided the strength of the wet material is more than 75 per cent of the dry one (**South African Bureau of Standards**, 1976). There are differences of opinion, however, between road authorities and there are also differences between the rock types. The latter are discussed in Chapter 8. The rock surface may be weathered since an oxidized layer does not affect the strength and durability of the material adversely. No washing or other means of cleaning are therefore required. These relaxations are permissible because the base material will eventually be fully embedded in the base layer and there are no demands with respect to adhesion and polishing.

The aggregate may be continuously or, in the case of a black base, semi-gap-graded, and the maximum size of the stone must not exceed 37,5 mm, to arrive at the required density, impermeability and stability. The fine aggregate is usually the crusher sand derived from the crushing process while the addition of foreign material should preferably be avoided. This is, however, not always possible because the quantity of fines produced during the crushing of different types of rock varies and, occasionally, a special effort is required from the crusher to produce the necessary fines. Although not acceptable to all road authorities, some of them have used 'soil' when it satisfied the design requirements in regard to plasticity and grading. Since the plasticity must be low (liquid limit not more than 25 and plasticity index not more than 6), the selection of such a fine aggregate is restricted to sandy soils whose -0,425 mm fraction must not contain more than 10 per cent of montmorillonite, while the kaolinite content can be 25 to 30 per cent. Highly disintegrated rocks, as they are encountered particularly in areas where N is more than 5, also yield durable fine aggregate, always provided the strength requirements can be met.

The strength of fresh rock, i.e. the coarse aggregate, which is of particular importance during construction when the material has to stand up to hauling, dumping and compaction, is largely a function of the intergrowth and size of the minerals in the case of crystalline rocks, and of the nature of the cementing matrix in the case of all other types of rock. In addition to these, the strength of a rock is determined by latent fissures which easily escape detection by visual inspection. They can of course be seen in microscopic slides and they may be the cause of unexpectedly low crushing values and especially of larger-than-expected differences between the results of the 10 per cent FACT on dry and wet samples. In general, coarse-grained rocks are less resistant to crushing than fine-grained ones, and the more similar the cementing matrix and the predominating mineral in sedimentary rocks are, e.g. quartzitic sandstone, the stronger is the bond. The strength of granular sedimentary rocks is also affected favourably by angular grains which provide for additional interlock. Not all rocks are equally easily crushed to the required shape and certain rocks are notorious for producing elongated or flakey chips. The hornfels which is generally known under the name 'Malmesbury shale' is such a rock and there are others especially those which are very hard or schistose. The problem can be overcome, however, by proper setting of the reduction rate of the crusher (**Shergold** and **Greysmith**, 1947; **Fulton**, 1977) in which case excellent stone can be produced from the strong rock, while schistose types, although perhaps of the intended shape initially, will tend to part along the planes of schistosity during construction or in service.

The problems which have been experienced in the past with cement-stabilized crushed stone bases have not been due to the natural material but to inadequate thickness and the shrinkage of the cement. There is no reason, therefore, to avoid cement as a stabilizing agent. Cement- or bitumen-stabilization will often be required for crushed stone bases especially to obtain the required bearing capacity. Certain types of rock contain sulphide minerals, such as pyrite, markasite or chalcopyrite, from which sulphuric acid and eventually sulphate salts develop when these minerals decompose (see Chapter 10). These decomposition products are, of course, deleterious to cement in the same way as organic substances may be

deleterious. If crusher sand is used for fine aggregate or for slurrying, the presence of sulphide minerals is even more dangerous because during crushing these minerals are largely separated from the rock, some of them may even be crushed themselves, and they are affected by decomposition at a faster rate than when they were still embedded in the stone surface. Such minerals, when fully embedded in the rock or when prevented from oxidizing, are harmless.

Natural gravel bases

The bulk material in the conventional base is weathered rock. In general, the same requirements as regards grading and plasticity as for crushed stone bases apply although, as shown by **Netterberg** (1971), the limits for plasticity can be relaxed for *calcrete*, particularly those types which exhibit self-cementing properties (**Netterberg**, 1975), and they may be as high as 15 for lightly trafficked roads but not more than 8 in highly trafficked ones. Rhodesian road authorities, however, are hesitant to go beyond 10 for any type of road and whatever the material. It can only be assumed at this stage that similar relaxations may apply to *ferricretes* some of which are also known to be self-cementing. Ferricrete with a plasticity index of 8 has performed satisfactorily in Rhodesian main roads, but it must be realised that the host material in such ferricretes has been quartz sand.

The strength requirements for the coarse aggregate are mostly such that the general tendency in the Republic of South Africa is to accept an ACV of 29 per cent or 110 kN obtained in the 10 per cent FACT on dry material, provided the strength of the wet material is at least 75 per cent of that of the dry material. It must also be pointed out that such generalizations always involve the risk that the values may be too low for certain materials and too high for others, and that the values actually used for selection depend further on the anticipated traffic. Rhodesian road authorities for instance are prepared to accept disintegrating, and certain pedogenic, materials, whose dry/wet strengths are as low as 50/40 kN in areas where the climatic N-value is greater than 5, in the bases of their lightly trafficked roads carrying fewer than 1 000 vehicles per day (**Mitchell**, 1971).

The use of weathered rock as coarse and fine material for the base requires fairly strict control of the stage of weathering which is the principal determinant of the material's durability. It has become known that weathering proceeds at a faster rate under a bituminous surfacing than in the local natural environment (**Clauss**, 1967), and therefore whereas a weathered material may initially be in a marginal condition which makes it still acceptable in regard to grading, plasticity and strength, continued weathering in the road may cause it to be in an unsuitable condition before the end of the road's structural design life. Excessive maintenance or early reconstruction are the consequences of such a lack of durability.

The severity of the effect of continued weathering under a bituminous surfacing depends on the environment and the type of rock. In all areas where N is more than 5, *disintegration* is the dominant form of weathering and the selection and control of the material must concentrate on its strength, particularly on its resistance to early degradation during hauling, dumping and rolling, but also to some extent under the traffic. It is strongly recommended that this strength be control-

led by crushing tests (the 10 per cent Fines Aggregate Crushing Test (10 per cent FACT) in particular, carried out on air-dry and on wet samples), and by specifying limits based on the strength of the dry material and the acceptable decrease of the strength of the wet material. The recommended limits are not the same for all types of rock (**Mitchell**, 1971; **Weinert**, 1968, 1969) and they will be given in the discussion of the different groups of road building materials (see Chapter 8).

Decomposition plays the most important role when the quality and durability of crystalline rocks, such as granite or dolerite, is assessed, particularly in areas where N is less than 5. Other types of rock predominantly *disintegrate* even where N is less than 5, except tillite, greywacke and volcanic blow-outs (tuff, volcanic breccia) which may experience disintegration or decomposition, depending on their composition. Crushing tests are again recommended for assessing the long-term strength of base materials, but in addition to these, crystalline rocks and those diamictites (e.g. tillite) (see Chapter 8) whose composition makes them liable to undergo at least partial decomposition, must be checked for the quantity of secondary minerals present before being used. The relation between the local N-value and recommended percentages of secondary minerals is shown in Figure 11. In this Figure, each material which plots left of the curves is suitable for the purpose represented by the curve concerned and that which plots right is unsuitable. Again, more detail will be given in the discussion of the different groups of road building materials (Chapter 8).

Attention must be drawn again to the possible presence of such deleterious materials as soluble salts or mica (muscovite) which may both have adverse effects on the base, although for different reasons which will be discussed in the chapter on deleterious minerals (Chapter 10).

In general, weathered rock can safely be used for the base, particularly in lightly trafficked roads, provided the necessary care is exercised to select material which has not yet weathered beyond that stage where continued weathering will eventually invalidate the design tests.

Natural gravel has to be stabilized in certain cases and the use of cement is mostly indicated. Material which calls for the use of lime because of the need for material modification (**Clauss** and **Loudon**, 1971) in addition to the desired strength, i.e. material with a clay content which would normally be considered too high for use in the base, may only be used in lightly trafficked roads if nothing else is available. In areas where fresh or weathered rock or other suitably strong material is very scarce or is absent, certain local soils may be used for the construction of roads which carry light to medium traffic. Bituminous materials, such as bitumen (**Marais** and **Freeme**, 1978) or the cheaper tar, or cement will have to be used in most such cases. Pure sand is a particularly suitable natural material for this type of bituminous stabilization, and this restricts the application of this method mostly to semi-arid and arid areas, i.e. areas where N is more than 5.

Concrete pavements

The materials requirements of the top layer of a Portland cement *concrete road*, although this layer carries the traffic, resemble those of the base of an asphalt pave-

ment rather than those of the bituminous surfacing. This is particularly so in regard to the aggregate which can be of a quality which is very similar to that of a good quality crushed stone base under a bituminous carpet. A concrete road has the advantage that it almost completely encloses the material in cement and thus protects this material much more from the ingress of water than can be achieved in a cement-stabilized base. Continued weathering, decomposition in particular, is also retarded by the high alkalinity of any moisture which might be present.

The distinction between the performance of the coarse and the fine aggregate is as important as in all other concrete work and reference should therefore be made to what applies in concrete technology (**Fulton**, 1977).

Since there is no distinction between surfacing and base in a concrete road, and since cement and rock are chemically more similar than bitumen and rock, adhesion in concrete roads does not pose such a serious problem as in bituminous surfacings. The surface texture of the aggregate is also less important (**Freedman**, 1971). The aggregate itself, however, is polished in the same manner as on the surface of a bituminous carpet, but in a concrete surfacing this does not normally pose a serious problem because the mortar is usually less resistant to abrasion than the aggregate. Coarse aggregate of a rather low polished stone value can therefore be used provided very little of the stone is exposed to the surface. The fine aggregate has a greater influence on the overall roughness of the concrete surfacing and its amount and grading are important. Depending on the mix design, between 30 and 40 per cent of coarse, hard natural sand or crusher sand, obtained from a low-polishing rock, should provide adequate skid resistance if a minimum texture depth of 0,75 mm can be maintained.

If, as is occasionally the practice, a bituminous layer is laid on the top of the concrete, everything said about bituminous surfacings applies to this cover.

SUBBASE

The subbase, the layer under the base, must provide the working platform for laying the base and it must help to distribute the load exerted by the traffic. The latter requirement determines the quality of the materials. A subbase may be stabilized with cement or lime, or it may be untreated, but the latter subbase should only be used under a base which is also untreated.

Since the stresses in the subbase are considerably less than those in the base, materials of considerably lower quality may be used, except when a cement-treated subbase underlies an untreated base. Rocks may be more weathered, disintegrated or decomposed and, although adequate grading is needed to achieve the required density, the permissible maximum size of stone after compaction (63 mm) is nearly twice that permitted for bases. The result is that the permissible variability of the material is greater than for the base, and suitable material can virtually always be found near the construction site.

Continued weathering can still affect subbase materials adversely and the percentage of secondary minerals in crystalline rocks, as well as certain diamictites, must again be considered in relation to the local N-value (Figure 11). The maximum permissible percentage of secondary minerals is, however, considerably high-
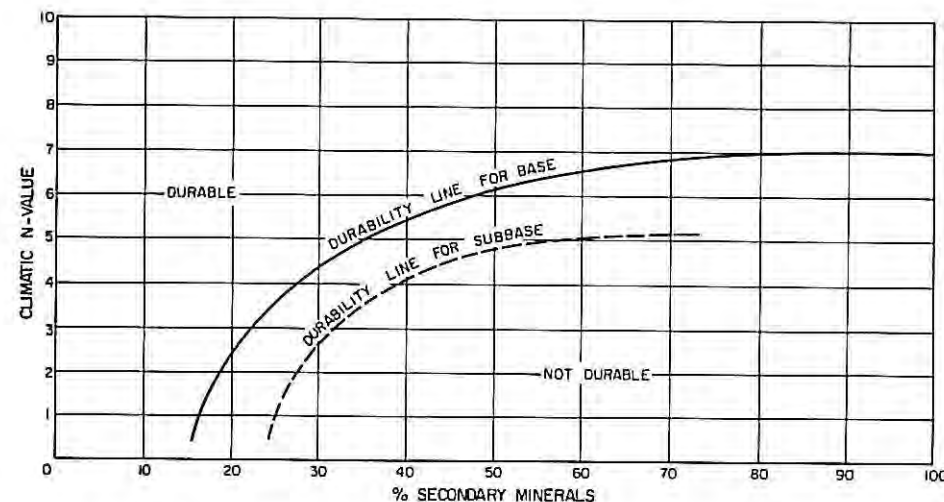
Figure 11: *The durability lines*

er than that for base material. Where borderline cases occur, the addition of lime or, if the design requires, cement keeps the soil moisture sufficiently alkaline to retard continued decomposition (**Clauss**, 1967). The higher permissible plasticity index, 10 or 12 depending on the percentage of the material passing the 2,00 mm sieve, also allows a higher percentage of clay, up to 20 per cent montmorillonite and 50 to 60 per cent kaolinite, in the -0,425 mm fraction. **Netterberg** (1971) has shown that the maximum plasticity index can be relaxed even more for calcrete – maximum plasticity index of 17 – and possibly also for ferricrete provided the host material is sandy.

In the subbase, use can be made of rocks which are subject to disintegration in all climatic environments, i.e. most sedimentary and all highly siliceous metamorphic rocks, and they can be considerably weaker than those required for the base. Use can now also be made of such rocks as weak sandstone and shale provided the shale complies with the strength requirements for subbase (see Chapter 8), and it does not contain excessive montmorillonite. Kaolinite is always acceptable and the usable quantity, which may be high, is controlled by the strength requirements. Except when using argillaceous rock (see Chapter 8), no crushing strength, Aggregate Crushing Value or 10 per cent FACT, need be specified and since, after compaction, the overall condition of subbase material approaches that of soil rather than that of rock, the design strength should be based on the CBR : minimum 45. Only in very dry areas, i.e. where N is more than 10, may the CBR be relaxed to 25 if no other material is available. Certain road authorities are reluctant to use the CBR to specify coarse subbase and prefer the Texas triaxial method of design (**McDowell**, 1954), for example Rhodesia who consider the Texas triaxial

class of about 3,2 as the best guide to strength.

Subbase materials which satisfy the design requirements in regard to bearing capacity, grading and plasticity may yet be suspect if they contain deleterious impurities such as sulphide minerals, e.g. pyrite, soluble salts, especially sulphates, and mica. Sulphide minerals decompose easily in the presence of air and water and first sulphuric acid and then sulphate salts develop. Both the latter can be harmful to cement- or lime-stabilized subbases and, if they can migrate upwards through the base, the salts may cause blistering of a bituminous surfacing. Mica, especially if it is muscovite, may be harmful to the compaction of the subbase. More details about these constituents are given in Chapter 10.

Stabilization of the subbase is often required especially under crushed stone bases. The choice of the stabilizing agent, lime or cement, depends on the composition of the rock used, and the presence or absence of clay minerals, montmorillonite in particular, is of special importance. Several of the natural materials used for subbase, particularly decomposing basic crystalline rocks and some argillaceous rocks, may absorb calcium if it is available. The amount of calcium which will be lost through this absorption prior to the onset of the calcium's cementitious action, i.e. the *initial consumption of lime (ICL)*, must be determined and the quantity of lime or cement required to give the intended effect must be adjusted accordingly (**Clauss** and **Loudon**, 1971). Although the theoretical background of this initial consumption of lime is not yet fully understood, it appears that it is related to the compositional instability of certain minerals. Such conditions occur especially in minerals which are decomposing into clay, and are apparently intensified when the process tends to produce montmorillonite rather than kaolinite. The conditions also occur in montmorillonitic shales, particularly when the latter contain an appreciable quantity of opaline silica ($SiO_2.nH_2O$).

The requirements for materials used in the *subbase* of a *concrete road*, i.e. the layer below the concrete surfacing, and those used in the subbase of an asphalt pavement are similar. Weathered rock may therefore be used but the material should be a bit stronger than that used in most flexible roads. To avoid pumping, the material should contain as little silt and clay as possible which means that the grading requirements must be stricter. Sandy soils or disintegrating rather than decomposing rocks, possibly with a plasticity index of less than 10, and a liquid limit of less than 40, should therefore be given preference. Cement or bitumen stabilization is definitely required for the subbase of major concrete roads, but it is not so necessary for residential streets especially if a strong, interlocking material can be used.

Gravel wearing course

The wearing courses of gravel roads consist of materials whose quality is comparable to that of the subbases of other pavements. It is obvious that oversize stones should be avoided because of the risk such stones provide to the moving traffic. Similarly, the material must be less clayey than is permissible for subbases because otherwise the road will become slippery, lose its shape and potholes may develop in wet weather. Too sandy a material, on the other hand, will cause corrugation.

The material of a gravel wearing course should be evenly graded (maximum grain size not more than 50 mm) and the material smaller than 4,75 mm should possibly not comprise more than two-thirds of the total. Nevertheless, material used for gravel wearing courses in dry areas, i.e. where N is more than 5, may be rather clayey to prevent corrugation and the development of dust, while less plastic material is preferable where N is less than 5 (**National Institute for Transport and Road Research**, 1980). Ideal materials are therefore evenly graded sandy soils or weathered rocks with a low to medium clay content, and calcrete or ferricrete – the latter two materials often lend themselves to the development of a 'blad' under traffic (**Netterberg**, 1971, 1978). This 'blad' is a smooth, slightly cemented surface which develops particularly if these pedogenic materials exhibit self-cementing properties.

The *dust nuisance*, so characteristic of gravel roads, is a result of the type of material required for the carriageway and the finer the material it contains or develops under traffic, the worse the nuisance is. The addition of sulphite lye provides a temporary solution as long as the weather is dry (**Fossberg**, 1966). Materials which possess self-cementing properties, e.g. certain calcretes and ferricretes, and develop a 'blad' are less dusty than the same type of materials without this property. Calcrete which is not self-cementing is the most dusty of all materials used for gravel road surfaces (*Plate 18*). Salt-containing soils are also less dusty than salt-free materials because, provided the air does not contain too much or too little moisture, the salts which migrate to, and accumulate at, the surface hydrate and exhibit slightly binding properties, thus causing another type of 'blad' formation. Such soils are known from the dry coastal areas of Southern Africa and the regional extent of their occurrence is rather limited. Since all salts are hygroscopic, these roads become excessively slippery if the atmospheric moisture rises above the level normal for the environment concerned, particularly during the early morning hours.

Most gravel wearing courses corrugate in time, particularly at places or in lanes where vehicles accelerate. Corrugation is particularly severe where a too sandy material has been used in areas where N is more than 5. Only where a 'blad' can be maintained or where salts bind the surface materials, is corrugation absent or at least suppressed. Pedogenic materials, if satisfactorily graded and not too strong or too weak, perform better in gravel roads than most other soils or weathered rocks.

SUBGRADE

The lowest layer of a pavement is the subgrade. This layer cannot be defined as clearly as those overlying it because it may consist of either the improved natural soil or of imported material. The purpose of this layer is again to provide a working platform and finally to spread the load of the traffic in such a way that no settlement occurs in the underlying untreated soil.

Natural sandy soil can usually be improved easily by compaction. The required depth of compaction varies and it must be deeper in pure sand, e.g. wind-blown or collapsing sand, than in a sandy soil which contains some natural binder. This

natural binder is mostly a clay component, and kaolinite is more suitable and is acceptable in a greater quantity than montmorillonite.

Clay soils require more careful treatment and the more montmorillonitic the clay is, the more care must be exercised. Grading and the mixing-in of lime will improve many in-situ clay soils sufficiently to obtain the minimum CBR of 10, if possible more than 10, and again kaolinitic clay is liable to pose fewer problems than montmorillonitic clay. There is, however, always the problem of thoroughly mixing-in the lime so that not too many lumps remain unaffected.

Expansive subgrades, i.e. those that are montmorillonitic clay soils, pose particular problems of treatment. These subgrades are wide-spread where N is less than 5 but are mostly localized where N is more than 5. Their expansion potential, however, depends on the field moisture content and the lower this moisture content is, the greater the expansion potential, i.e. it generally increases with increasing N-value. One way of treating such clays is to prewet them so that the field moisture content is deliberately increased almost to saturation point. This method has been applied to isolated spots such as culvert sites but so far only once experimentally to some length of road (**Construction in Southern Africa**, 1975). Another possibility is of course lime stabilization of the subgrade which has already been mentioned above. Partial or complete removal of the clay would probably be the most efficient approach. The removed material would have to be replaced by non-expansive material, however, and this method is therefore only worth considering if such replacement material is economically available. In all cases, except when the expansive subgrade can be removed completely, the pavement must be designed so that sufficient overburden is provided to counteract any remaining expansion potential.

Subgrade material must also be imported where the carriageway has to be elevated more than could be achieved by the base and subbase alone because it is necessary to keep the surface about 1 m above the water-table and because of the danger that the road may be flooded during or after heavy rain, as is the case in most of the flat Karoo areas. The same applies to the filling-up of surface irregularities to obtain a level road surface. In all these cases, subgrade material has to be imported from a selected source.

Selected material for subgrade or fill (a subgrade of more than average thickness) is always soil or highly weathered rock. Fresh rock may be used in a fill whose material has been obtained from a nearby cutting. The quality requirements of such a material are much lower than those for the material of any other layer of a road structure and considerable play is permissible in regard to grading and plasticity provided these two are kept in a certain balance, i.e. a bulk material with a low soil mortar content can be more plastic in its -0,425 mm mesh fraction than another material with a higher soil mortar content. No strength limits are required except with regard to the bearing capacity (CBR) of the compacted overall layer and there is no need to consider durability.

Only a few precautions are required:

1. Kaolinite is a favourable component since it provides the material with a practically non-expansive binder. Montmorillonite, as in all layers of a pavement, must be avoided, or at least it should not exceed 30 to 35 per cent of the -0,425 mm fraction of the soil mortar in subgrade material.

2. Soluble salts may be a problem especially in dry areas where N is more than 5, and more so when N is more than 10, when the country soil is salty and there is therefore an unlimited supply of such salts. Such a material cannot be improved and the necessary precautions must be taken in the design.

3. Mica (muscovite), which occurs in large quantities in certain sands and in weathered granite, gneiss, mica schist and phyllite, causes problems with compaction because of the 'spring action' of the muscovite minerals which may prevent achievement of the intended density or, even if it is achieved initially, can cause rutting in the compacted layer at a later stage. Micaceous soils are best stabilized with cement.

Embankments

Embankments, some of which may be several tens of metres high, require as careful a design and construction as any other large structure. Although almost any material could be used, precaution is necessary to provide proper drainage and to prevent erosion (**Harmse** and **Van Zyl**, 1975), settlement or heave. The construction material is mostly obtained from nearby cuttings or, if they do not provide the required quantity, from borrow pits. Good quality material should be used in the bottom metre or so and material of the same quality as the selected subgrade to be used in the envisaged type of road in the top metre below subbase level. In addition, if the embankment is to be built on expansive soil which cannot be removed, such soil must be covered, after clearing the vegetation, by a sand blanket for at least one year prior to the beginning of the construction in order to stabilize the soil moisture conditions.

The condition of the materials in cuttings and borrow pits usually improves with depth. This means that the type of material required in the bottom portion of the embankment is usually only reached after the removal of considerable quantities of inferior material which could well be used for later construction stages. Careful planning of dumping and hauling is therefore required.

The material between the bottom and the top metre of the embankment may be almost any type of rock or soil, and blocks and boulders of solid rock obtained from cuttings are often used. The permissible size of the solid rocks depends mainly on the thickness of the lifts which are compacted in one run. The dimensions of the rocks must of course be slightly smaller than the dimensions (thickness) of each compacted layer to ensure that such rock blocks are fully enveloped by soil. Since the maximum thickness of such layers is hardly ever more than one metre, the biggest rock blocks should have a diameter of not more than 900 mm (**National Institute for Transport and Road Research**, 1978b).

STABILIZATION

Stabilization has been mentioned several times in the discussion of the various layers of a pavement. Generally, stabilization is any process which results in the improvement of the properties of a material. Stabilization of road construction ma-

terials can be done mechanically by simply mixing two materials, or chemically by adding a reactive substance.

Stabilization with soil

This type of stabilization can be applied to a cohesionless material by blending it with a clayey soil, or to a very cohesive material by adding sand, and so adjusting deficiencies of grading. By adding a clayey soil to a cohesionless material a natural binder is also supplied. Soils whose clay component is kaolinite are preferable to those which are largely composed of clay minerals of the smectite group, such as montmorillonite. This implies that a decomposed or highly decomposed acid crystalline rock is more suitable than a decomposed or highly decomposed basic crystalline rock. The addition of sand to a very cohesive soil helps to reduce the overall plasticity of the structural layer concerned. Although this is seldom done nowadays, the method has been applied successfully in the past to lightly trafficked roads.

Stabilization with bituminous materials (black bases)

The use of bituminous materials is a form of stabilization where a manufactured material is added to a cohesionless material and, in Southern Africa, this has been applied successfully to wind-blown sands in areas where N is more than 5. The stabilizing agents are usually bitumen (**Marais** and **Freeme**, 1978) and occasionally, especially in Rhodesia, tar.

Bituminous materials may also be used for the stabilization of continuously or gap-graded aggregate, natural or crushed, in the bases of roads which carry heavy traffic (**National Institute for Transport and Road Research**, 1978c). The decision to use such bases, which are comparatively expensive, depends mainly on the economics of construction which are guided by the availability of suitable materials.

Stabilization with lime or cement

Stabilization which aims at preserving, modifying where necessary, and cementing a material is achieved by adding manufactured stabilizers which react chemically occasionally with the soil components but mostly with themselves in the presence of water. Such materials are lime $(Ca(OH)_2)$ and various types of cement.

Preservation is a process by which continued decomposition is retarded by keeping the soil moisture alkaline (**Clauss**, 1967). Only 0,3 per cent of lime or cement in a structural layer are required for this purpose (**Clauss** and **Loudon**, 1971), and since this quantity is far less than that used in any stabilization project, preservation is never carried out as a separate task. Materials which benefit from preservation are decomposing crystalline rocks and occasionally diamictites (see Chapter 8) of borderline quality.

Modification is a process by which the plasticity of a material is reduced or which could even make a plastic material non-plastic (**Clauss**, 1970). Lime $(Ca(OH)_2)$ is the best stabilizer for this purpose since the modification is the result of an interaction between the clay minerals and the lime in its ionised form: Ca^{++} and $(OH)^-$. Modification takes place shortly after the lime is added, but strength is gained much more slowly provided that not too much lime is consumed by certain soil constituents before any cementicious effect can be obtained (**Clauss** and **Loudon**, 1971). All clayey soils, many disintegrated argillaceous rocks and decomposing crystalline rocks and most diamictites (see Chapter 8) are materials which may benefit from modification.

Cementation may well be achieved by using lime but takes place much faster with cement. The cementicious effect is achieved by the fast hydration and setting of the cement in the presence of water. The hydrated crystals so formed bind together the soil particles; there is hardly any reaction between the cement and the soil components, although some calcium may initially be absorbed by certain soil components while the cement is in a state of hydration and setting (**Clauss** and **Loudon**, 1971). Consequently it is obvious that granular, non-plastic materials like gravel and sand, or disintegrated or crushed rocks benefit more from cement than from lime stabilization. A particular use of cement has been developed in the United States. It has become known as *soil cement* and for details reference should be made to the pamphlets and manuals which are published from time to time by the Portland Cement Association (PCA).

There are numerous other stabilizing agents, e.g. various types of salt and resin, which are used occasionally. Their use is limited, however, often restricted to particular environmental conditions, and reference should be made to **Ingles** and **Metcalf** (1972) for more detail on this subject which lies outside the scope of this book.

CHAPTER 7
TESTING THE DURABILITY

GENERAL

It should be clearly understood that this chapter is not a testing manual. The numerous tests for road building materials have been specified by the different road authorities, the South African Bureau of Standards and also the Standards Association of Central Africa and reference is made in Appendix 4 to their and overseas publications for details of test methods. Only methods which have never been standardized elsewhere will be described in more detail.

Whenever a material is to be used for some purpose, its suitability must be ascertained. In many cases this assessment is based simply on previous experience. If the material concerned is artificially made, for a particular purpose or a set of purposes, e.g. cement or bitumen, such a decision based on previous experience is feasible in many or perhaps in most cases. In the case of a new application, however, previous experience can only be used as a guide while a testing programme is required for the determination of the precise requirements and properties of the material. Based on such test results, a completely new material may be designed or the manufacture of an existing one may be so amended that it satisfies the new requirements.

The advantage of artificial materials is thus that their production can be controlled, that they can be made to suit a purpose and that their reactions in use can be predicted. In this regard, all man-made materials differ basically from natural ones.

An intermediate position between artificially made and natural materials is probably occupied by certain industrial by-products such as slag and fly-ash. Although they are 'artificially made', their composition is incidental rather than controlled and their engineering properties are similar to those of certain extrusive rocks and materials such as pumice, or volcanic ash or tuff, while their normally low silica content makes them comparable rather to basic or even ultra-basic rocks.

Natural rocks and soils are not made artificially and they must be accepted as they are. Rocks exposed to the surface of the earth change gradually into soil, passing during this transformation through many intermediate stages during which their engineering properties may be very different. The resulting soil type depends on the parent rock, the mode and rate of this transformation and the environment in which it occurs. Neither rocks nor soils are therefore static entities. Most of the materials in a road, however, are obtained from natural sources and because their conditions can vary sometimes over a distance of only a few metres, these conditions must be monitored continuously.

The number of tests for the determination of the properties of rocks and soils is large. Each of these tests usually measures only one property or perhaps a few related properties. Whichever test is applied, however, it is applied on the supposition that the measured property does not change during the geologically short structural design life of a road. The numerous tests can be divided into two groups, namely those which determine a natural condition of a material (e.g. its plasticity or in-situ bearing capacity), and those which determine either the same or another property after some form of treatment has been applied to make the material answer a specified requirement. Whichever type of test is applied, however, one should always bear in mind what is actually being measured. Thus, when the plasticity index is determined, in reality it is only a measure of the water-retaining capacity of a soil within two arbitrarily set limits. All other information derived from this test will be in the form of interpretations based on general knowledge of soils and on previous experience; indiscriminate generalizations should not, however, be made. Netterberg (1971) has shown, for instance, that the plasticity of calcrete in the layers of a pavement can be higher than that of clay soils or decomposed rocks because the mineralogical and physical properties which cause plasticity are not the same in these materials. The same phenomenon also applies to other pedogenic materials.

Since at some stage of weathering rocks and certain soils, both residual and transported, may change noticeably during the design life of a pavement, the results obtained from repeated tests may also change. There are of course natural road building materials which are entirely stable over the service period of a road. These are in particular fresh rocks and those soils which are in reasonably good equilibrium with their environment.

Fresh rocks are stable road building materials although they definitely change through weathering. The onset of weathering, particularly decomposition, in a fresh crystalline rock in an environment where the N-value is less than 5 becomes noticeable after about ten years when the development of an oxidized and hydrated surface layer is unmistakable; ten years is about half the normal structural design life of a road. In an environment where N is more than 10, this process is much slower and no such surface changes will be noticed after this period. Although in a suitable environment a definite change in the surface of such a fresh rock can be noticed after ten years, most of these changes have no effect on the engineering properties of the rock which can be regarded as unchangeable during the whole life of a road; an exception is the oxidizing sulphide minerals, e.g. pyrite, which may cause serious problems (see Chapter 10).

Similarly, a soil which has developed over a sufficiently long time and which is therefore in near-perfect equilibrium with its environment does not experience during the life of a road changes which would be detrimental to the structural strength of the pavement.

Between those conditions which are colloquially termed 'rock' and 'soil' lies a transitional condition which covers all stages of the actual weathering of rocks. These weathered or rather weathering materials, are, however, those in which the road engineer is particularly interested because, for reasons of economy and workability, they have to provide the bulk of the materials in a pavement. Weathering, both disintegration and decomposition, proceeds at an exponential rate up to a certain point, which varies among the different types of rock, after which the process probably slows down until a type of residual material is approached which is in the best equilibrium with the prevailing environmental conditions. The condi-

tion of weathering (weathered) rock which is used for the layers of a pavement or for gravel wearing courses, is somewhere on this curve of weathering. The removal of such a material and its laying in a road does not halt the process of weathering, but rather this process is speeded up due to handling, the increased temperature, particularly in the base which may be more than 50 °C near the contact with the surfacing, and the permanent presence of some moisture due to a waterproof cover of a bituminous or asphalt surface. Some change in the quality of a natural road building material after laying must be accepted as unavoidable. The material must not, however, change to such an extent that it does not stand up to the design specification throughout the life of the road. Any road building material is used with the assumption that its condition, as determined by numerous tests, may perhaps change a little but not so much that it will cause failure.

Failures of roads in the Republic of South Africa during the fifties and early sixties, all of them occurring in an environment where N was less than 5, e.g. near Ladysmith in Natal, near Bethlehem and Queenstown, confirmed that such changes do occur in natural road building materials although these materials were accepted and used after the usual testing. The failures have shown further that the usual tests, such as Atterberg Limits, CBR and others, do not necessarily reveal all there is to know about the quality and durability of a natural road building material (*Plate 19*).

DETERMINATION OF DURABILITY

A durable natural road building material is in such a condition at the time of laying that it can be assumed with reasonable certainty that the unavoidable change will not during the life of the road transform it into a condition which leads to failure. A material may of course be durable for one purpose but not for another. Many shales, for instance, might make sufficiently durable selected subgrade because considerable break-down is tolerable in a subgrade, but they would change quickly into a useless silty clay in a base. Durability is therefore not an absolute property of a natural road building material but is a function of the condition of such a material at the time of testing in relation to the intended use, as e.g. for base, subbase etc.

There are a number of properties which influence or determine the durability of a natural road building material. They have been described in more detail in the 8th Chapter because the classification of Southern African natural road building materials has largely been based on them. Durability cannot be tested directly, however, because it is actually a supposition about the permanence or change of the present condition of a material in the future once this material has been subjected to some treatment and endurance. This supposition must therefore be based mainly on an understanding of the physical and mineralogical properties of the material. A number of tests, especially if carried out on the material in its natural condition, can then be used to confirm the assumed durability or to detect hidden weaknesses. The aim of such tests should be to determine the condition of a natural road building material at the time of testing which will allow sufficient flex-

ibility, so that the unavoidable changes will not render the material unsuitable before the expiry of the design life of the road.

The durability of natural road building materials depends mainly, although not entirely, on the stage of weathering which a material has attained at the time of testing. If weathering has proceeded too far, its continuation will render the material unsuitable before the expiry of the intended service period of the road, even if the selection and design tests indicate that the material is acceptable. The prevailing type of weathering, disintegration or decomposition may have reached such a stage that there is no allowance for the unavoidable deterioration in the quality of the natural road building material during the service of the road.

Durability is therefore the first property which must be determined when selecting natural materials for any road construction purpose because all the other tests will be useless if the information they yield about other properties of the material will cease to be valid before the road has given the intended service.

DETERMINATION OF THE DEGREE OF DISINTEGRATION

Physical tests are the obvious means of measuring the degree of disintegration and particularly of detecting hidden weaknesses in a rock. All these tests measure the strength of naturally disintegrated or artificially crushed rock within certain specified limits of grain size. They should not be confused with strength tests intended for rock masses, as e.g. the determination of the unconfined compressive strength. A number of tests for determining the strength of rock aggregate are available.

The sulphate soundness tests

The sulphate soundness tests, which use the crystallization force of either sodium or magnesium sulphate ($Na_2SO_4.nH_2O$ or $MgSO_4.nH_2O$ respectively) to make a rock disintegrate, also carry the unfortunate and misleading name 'weathering tests'. This form of disintegration, however, occurs in nature only under very special conditions and it can be expected to occur on a larger scale only where N is more than 10. One particular process of natural disintegration is thus used for the measurement of the physical strength, and it is considered indicative of the weatherability of a rock. The tests employ a process which is deliberately guarded against in road construction: the action of highly soluble salts. This in itself could be accepted since the salts are only used as a tool to break the rock and the test result is interpreted in terms of the strength to resist physical break-down. These tests are, however, carried out on materials which are so variable that they require different numbers of test cycles to obtain an interpretable result. This does not allow the direct comparison of different types of rock. Moreover, the tests are time-consuming, relatively expensive and probably also poorly reproducible.

The sulphate soundness tests are no longer used frequently in South Africa and the South African Bureau of Standards and the Department of Transport have omitted them from their test specifications. The effect of sodium sulphate on rocks is slightly less vigorous than that of magnesium sulphate, and test specifications are usually based on the former salt. In the United States, for instance, a

loss of 10 to 15 per cent by mass after five cycles using sodium sulphate is usually specified for the different road-making aggregates, most limits being 12 per cent. Slightly higher losses are permissible when the test is done with magnesium sulphate (**American Association of State Highway and Transportation Officials,** 1974).

Freezing and thawing test

The freezing and thawing test, where ice plays the role of the growing crystals, is similar to the sulphate soundness test and the same comments apply. This test also aims at measuring the resistance of stone to the natural force of ice which is very common in large parts of the world, especially in many highly developed countries. In Southern Africa, however, prolonged frost occurs only on the highest mountain ranges and in the highlands of Lesotho.

Wetting-drying test

The wetting and drying test is perhaps less artificial than those tests which use the growing force of crystals because they measure only the reaction of a material to the controlled alteration of its moisture content. The results of the wetting-drying test depend on the composition of the test sample and the reaction of its components to moisture variations. Thus, growing crystals (if soluble salts are present), expansive clay minerals and the solution of cementing materials may all cause the break-down of one type of rock while another type may be cemented as a result of the reprecipitation of dissolved materials, as e.g. certain carbonates or compounds of iron. This test, which is intended to yield information on the possible degradation of materials, may therefore also have the opposite effect and the strength of certain materials may increase.

The latter process has become known as *self-stabilization* or self-cementation (**Nascimento** *et al*, 1963; **Aitchison** and **Grant,** 1967; **Grant** and **Aitchison,** 1970; **Netterberg,** 1975) and it was originally investigated more closely by **Nascimento** and co-workers. They observed that certain oven-dried soil pats used in the shrinkage limit test did not break up entirely when immersed in water. In order to study this in more detail, soil pats prepared for the shrinkage limit test, after oven drying, were allowed to absorb water. The pats were placed on an uncoated porcelain dish and the latter was then immersed in water up to its top. After 24 hours some samples had broken down completely, having absorbed water well in excess of the liquid limit, while the amount of water absorbed by others was between the liquid and the shrinkage limits, almost equalling the shrinkage limit in some cases.

The quantity of water absorbed after 24 hours was called the absorption limit and it appeared that in all samples which absorbed less water than the liquid limit, some water-resistant material had precipitated during oven drying. This process was called 'petrification' (**Nascimento** *et al*, 1963) and was then defined as

$$\text{Petrification degree} = \frac{\text{shrinkage limit}}{\text{absorption limit}}$$

The closer the petrification degree is to 1, the more beneficial cyclic wetting and drying may be under laboratory conditions as well as in a seasonally wet and dry climate.

A similar effect has been observed when determining the *soaked CBR* after the sample has been subjected to a number of wetting and drying cycles. The CBR decreases in certain cases and increases in others and again the only possible explanation appears to be the precipitation of originally soluble components in a less soluble form, e.g. ferrous iron precipitated as ferric iron. **Netterberg** (1975), in a well-documented discussion of the various opinions on self-stabilization, has suggested that five cycles of one day wetting and one day drying 'is probably the most reliable and easy to interpret of the tests' available so far.

Wetting and drying does therefore not always lead to the degradation of natural road building materials for the opposite may occur as well. Although apparently not confined to them, self-stabilization has so far been observed mostly in materials such as laterite, ferricrete and calcrete. Basically, all the above tests were designed to measure the potential degradation of natural road construction materials despite the fact that the wetting and drying test may also produce results to the contrary. It is then a common feature of these tests that natural materials of different strengths, or of the same type but at different stages of weathering, require different numbers of test cycles. The number of these cycles must be large if a strong material is tested because nothing or almost nothing may happen during the first few cycles and the degree of disintegration may then be too insignificant to be measured accurately. The number of cycles must be small for weak materials to prevent complete disintegration prior to the completion of the test. As a compromise, five cycles appear to be most favoured. All these tests are time-consuming, more so if done on strong than on weak materials, and moreover **Tubey** and **Beaven** (1966) have shown that there is a good correlation between the sulphate soundness test and the 10 per cent Fines Aggregate Crushing Test on dry and wet materials.

When applied to measure the strength of rock pieces, all the above tests are actually 'mild crushing tests' and the same information can be obtained from any of the proper crushing tests which can be carried out much faster.

Aggregate Crushing Test

The most generally known of the crushing tests is the Aggregate Crushing Test, which yields the percentage known as the *Aggregate Crushing Value* (ACV). This value is obtained by applying a load of 400 kN to a sample crushed to pass the 13,2 mm sieve and retained on the 9,50 mm sieve, and by determining the percentage of those fines produced during the crushing process which pass the 2,36 mm sieve. The load of 400 kN must be obtained by gradually increasing the force applied over ten minutes. The test is not very sensitive on weaker material and it should therefore be used only on rock which is sufficiently fresh and strong not to produce a compressed lump in the test-mould before the full specified load of 400 kN has been applied. If this happens to a sample at a load of less than 400 kN, any further increase of the load is unlikely to produce more fines.

Figure 12: *Graphical determination of the 10% FACT for a number of materials*

In 1959, **Shergold** and **Hosking** amended the Aggregate Crushing Test by determining at which load 10 per cent of fines passing the 2,36 mm sieve would be produced. This test allows the load to be varied while the quantity of fines produced is fixed. It has become known as the 10% Fines Aggregate Crushing Test (10% FACT). The sample preparation and apparatus are the same as those used for the Aggregate Crushing Test, and the load required, which is of course variable here, is again obtained by gradually increasing it over ten minutes. The load varies according to the condition of the sample so that only 10 per cent fines passing the 2,36 mm sieve are produced and the required force, and not the percentage of fines, is measured and used for interpretation.

Although an experienced operator can make a considerably accurate estimate of the load required to produce about 10 per cent fines and increase the load accordingly during the ten minutes of the test, the exact value will seldom be obtained directly. It is sufficient, however, to obtain between 7,5 and 12,5 per cent fines in the actual test. The load required to obtain the specified 10 per cent fines can then be calculated from the formula (**Shergold** and **Hosking**, 1959):

$$10\% \, \text{FACT} = \frac{14\,x}{y+4}$$

where x is the load in kN giving y per cent fines, y being between 7,5 and 12,5 per cent. Another possibility is to do three determinations, varying the load three times and using a new sample each time, and then to find the required force at the intersection of the curve through these three test results with the line for 10 per cent, as shown in Figure 12. In this Figure, the extensions of the curves through the test results intersect the x-axis at imaginary negative values of the percentage of fines. Most of these intersections are in the vicinity of y = - 4 which value has then been used in both the numerator and denominator of the above expression. The deviations from the exact intersection with y = - 4, which will be found in most cases, have little effect on the computed results (**Shergold** and **Hosking**, 1959).

The 10% Fines Aggregate Crushing Test has the great advantage that it can be applied to all materials, however weak, as long as pieces between 13,2 and 9,5 mm diameter can be obtained, thus allowing the direct comparison of strong and weak materials and all transitions between them.

The 10% FACT is therefore particularly recommended for the determination of the crushing strength of stone and for information on the degree of disintegration of a rock.

The 10% FACT should always be done on both air-dry and soaked samples. The portion of the total sample which is intended for the wet test must be soaked in water for 24 hours. Thereafter the sample is drained until the flow of free water ceases and it is tested immediately by the same method as that used on the air-dry sample. For the sake of interpretation, a value is specified for the force required to produce 10 per cent fines from the dry materials while the value obtained from the
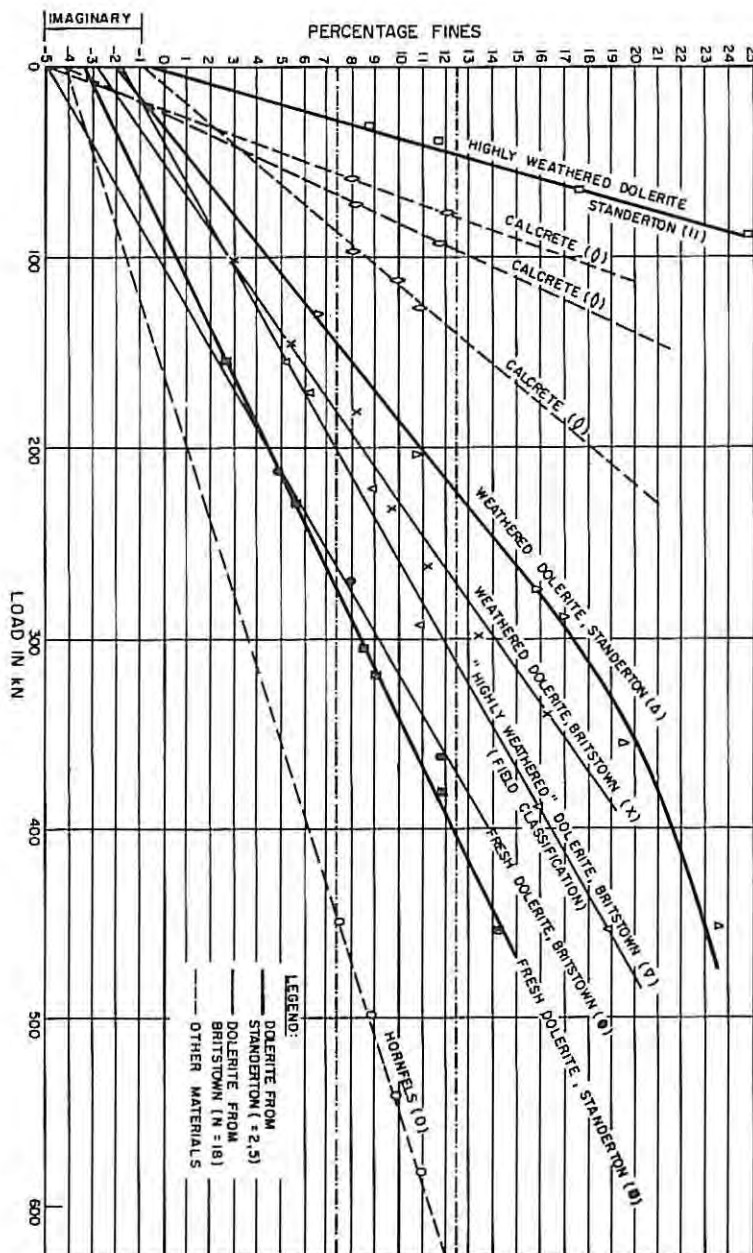
wet test must not fall below a specified percentage of the dry test results. The specified dry values and the permissible decrease of the wet values vary with the type of rock tested and they are given in the discussion of the properties of the groups of natural road building materials in Chapter 8. The crushing strength of a wet rock is almost always less than that of a dry rock; a wet rock therefore provides a better indication than a dry one alone of the presence of fine cracks and other, often obscure, weakening properties. It also reveals more of the resistance of the rock to construction operations, rolling in particular, which are always done in the presence of water.

Relation between ACV and 10% FACT

It is obvious that the Aggregate Crushing Test and the 10% FACT measure essentially the same property of a rock, namely its strength or, more precisely, the crushing strength of broken rock. Unfortunately, these tests have often been said to measure the hardness of stone. This is, however, not entirely true since the strength which is actually determined is not exactly the same as the hardness, i.e. the resistance to scratching or abrasion, of the rock or its minerals. Both tests are very similar and it is therefore possible to estimate the results which would be obtained in one of them from those obtained in the other. The relationship of the test results (Figure 13) can be represented by the equations

$$ACV = 40 - 0{,}1083 \,(10\% \text{ FACT}) + 0{,}000\,083 \,(10\% \text{ FACT})^2$$

or

$$10\% \text{ FACT} = 650 - 28{,}33 \,(ACV) + 0{,}33 \,(ACV)^2$$

where ACV is expressed in per cent and 10% FACT in kN. The above equations apply only when the ACV is less than 30 per cent and the 10% FACT more than 100 kN. The strength of road construction materials, however, is seldom less than 14 per cent.in the ACV or more than 300 kN in the 10% FACT. In the strength range between 14 and 30 per cent for the ACV or 100 to 300 kN in the 10% FACT, the relation is almost linear and the following, less involved, equations can be used as well:

$$ACV = 38 - 0{,}08 \,(10\% \text{ FACT})$$

or

$$10\% \text{ FACT} = 12{,}5 \,(38 - (ACV))$$

where again ACV is expressed in per cent and 10% FACT in kN. The results obtained from the above equations are correct within ± 1 per cent for the ACV and ± 10 kN for the 10% FACT in most cases. In the ranges of greater strength, especially if the ACV is less than 8 per cent or the 10% FACT greater than 450 kN, the correlation of the two tests becomes increasingly less accurate. This is partly due to the particle shape whose influence increases with increasing strength. The main reason, however, is the simple fact that there may be a material from which no fines could be produced with a force of 400 kN (ACV of 0 per cent) while 10 per cent fines could well be produced under different, very much greater forces. In this case, the curve through the test results coincides with the ordinate. Similarly, in the direction of the abscissa the curve through the test results cannot exceed an ACV of 100 per cent.
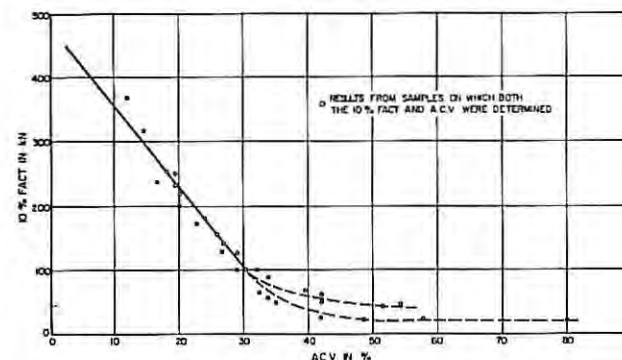
Figure 13: *Relation between ACV and 10% FACT*

The ACV becomes unreliable when it is greater than 30 per cent and, consequently, the above formulae only apply when the ACV, measured or calculated, is less than this value. Whenever the ACV is greater than 30 per cent, the 10% FACT has to be used for the determination of the crushing strength. However, the 10% FACT on dry and wet material is the more revealing of the two tests under all circumstances.

Both crushing tests are of course done best in a laboratory. There are a few other tests, however, which can be used easily in the field and which may be applied successfully to similar ends if their results are interpreted correctly.

Treton Impact Test

This test is also known briefly as the Treton Test and it is somewhat similar to the British aggregate impact test (BS812:1975). It uses the impact of a falling mass to measure the crushing strength of rocks. The test is performed on selected angular and cubical chips which pass the 19,1 mm sieve and are retained on the 16,0 mm sieve. Fifteen to 20 chips weighing as nearly as possible 50 times the relative density of the stone are required. The relative density of the stone can be taken as 2,70 and it is therefore not necessary to determine it in each case since about 135 grammes will be required in most cases – the only exception being stone of the group of Metalliferous Rocks which is described in Chapter 8. These stones are placed on the steel disc or anvil of the test cylinder so that they form a single stone layer. The chips must always be so selected that they are of more or less even height and their tops therefore approximately at the same horizontal level. The anvil rests on a steel base plate and over it is placed a hollow steel cylinder 197 mm high with an internal diameter of 101,6 mm. (These somewhat strange measures have been derived from the original imperial measures of 17¾" and 4" respectively.) The chips are given ten blows with a cylindrical steel hammer, with a mass of 15 kg (33 lbs). The bottom of the plunger should be in line with the top of the cylinder when dropped.

After the test, the broken chips are screened through a 2 mm sieve and the percentage by mass passing this 2 mm sieve is calculated. The Treton Impact Test must always be carried out in triplicate and the average value is taken as the Treton value of the stone.

The Treton Impact Test has the advantage that its execution does not depend on a well-equipped laboratory. The test can be carried out in the quarry, in a borrow pit or at an outcrop as long as a sufficiently large number of correctly sized and shaped pieces of stone is available. A wet test, if required, could be done in a field camp.

Values below 15 per cent indicate tough aggregate which is suitable for surface treatment work. Values between 15 and about 20 per cent are obtained from aggregate which is suitable for bituminous mixtures, and values up to 32 per cent indicate aggregate suitable for mass concrete. Unfortunately, no other specific values are available but standards suiting a particular job can be set before starting a material survey, by comparing the results of the 10% FACT on dry and wet material with the results on the same material obtained from the Treton Impact Test in the laboratory.

In South Africa, the Cape Roads Department uses the Treton Test as a control if the Aggregate Crushing Test on dry material gives borderline results, e.g. between 29 and 31 per cent. If the Treton Test on dry material gives then more than 40 per cent, the material is not used.

Aggregate Pliers Test

This test was invented by **Netterberg** (1971, 1978) during his study of calcrete. It is almost ludicrously simple and it can be carried out by any road worker who has a knowledge of simple arithmetic using a pair of standard pliers. The test offers possibilities for determining the strength and simultaneously the degree of disintegration of a stone; its application is restricted, however, to relatively weak materials.

The method of test involves obtaining 100 to 200 pieces of the air-dry 19,1 mm to 12,7 mm aggregate portion of the material. The first step is to try to break the pieces between the thumb and forefinger, using both hands simultaneously. The unbreakable pieces are then tested with an ordinary pair of 180 mm pliers in the concave serrated portion of the jaws. In both cases more or less maximum strength should be exerted, although this aspect does not appear particularly critical. The total percentage passing the fingers is called the Aggregate Fingers Value (AFV) and the percentage passing the pliers the Aggregate Pliers Value (APV) and the test as a whole is called the Aggregate Pliers Test.

A fair correlation between the Aggregate Pliers Test and the 10% FACT has been found to exist. The material unbroken by the pliers has a 10% FACT value of more than 100 kN while that unbroken by the fingers has a value of more than 50 kN. The applicability of this test is limited to material whose crushing strength is less than about 150 kN.

Summary on crushing tests

The common feature of all tests discussed so far is that the whole or part of the rock is crushed. Because of the nature of these tests, except the Aggregate Pliers Test, fissures which were not there initially may develop in the sample pieces while the test is in progress. Such newly formed fissures may of course influence the test result. There is no control over this but it can be assumed with reasonable certainty that the tendency to develop such fissures is an inherent property of the rock concerned and that this will also have an effect during the construction of a road.

The disadvantage of all tests which use the growing force of crystals is the varying number of test cycles required for strong and weak materials. The disadvantage of the Aggregate Crushing Test is its confinement to strong material and that of the Aggregate Pliers Tests that it can only be applied to relatively weak materials. Only the 10 per cent Fines Aggregate Crushing Test and the Treton Impact Test are suitable for strong and weak stone alike but their application is limited to coarse aggregate.

Testing in a laboratory can, of course, be carried out with greater care and better control over all variables than testing in the field. This is the reason that the 10% FACT has received so much attention and that it has been considered a particularly suitable means of predicting the durability of the strength of natural road building materials. Another advantage of the 10% FACT is that it is equally suitable for testing dry and wet material which is not the case with the Aggregate Crushing Test. This is so because in this test the percentage of fines is fixed and the quantity measured is the load, however small. Consequently, the sample in the test cylinder is never compressed so much that it eventually forms a dense lump. Reference to the 10% FACT has therefore been made throughout the discussion on the physical durability of natural road building materials in Chapter 8.

Compression tests

Compression tests are similar to crushing tests in that the sample is subjected to a force, which is now increased, however, until failure occurs. The definition of the moment of failure requires that the test be carried out on a single piece of sample, always shaped to suit the test apparatus, and not on a larger or smaller number of such pieces which may interact during the test. The commonest shape of samples for compression tests is cylindrical with a diameter of 150 mm and a height of 200 mm, but other sizes are used as well. When tested, these cylinders stand on their basis and the pressure is exerted vertically in the direction of the long axis.

There are basically two different types of these tests which differ in regard to the lateral pressure employed.

In the *triaxial compression test*, different constant lateral pressures up to about 150 kPa, usually obtained by air or hydrostatic pressure, are applied while the sample is compressed. Consequently, a number of similar samples have to be tested, each with its own lateral pressure, and a sufficiently large sample must therefore be obtained for the preparation of the required number of test cylinders. The

use of lateral pressures makes it obvious that this test has been designed primarily for soils. In most cases, the tested soil has to be prepared in some form and the test does, therefore, not actually inform on the truly natural conditions of the sample.

In a *uniaxial compression test*, occasionally also called an 'unconfined compression test', the sample is compressed without employing a lateral pressure. The uniaxial compressive strength (UCS) can of course be determined for a soil as well as a rock and, in fact, the first one of a series of triaxial compression tests of soil samples is usually an unconfined one. The uniaxial compression test is again carried out on cylindrical samples and the equipment used is similar to that for the triaxial test, without the lateral pressure. Soils may be tested in this way treated or untreated, while rocks are always tested in the natural condition, except that they are shaped cylindrically.

It is obvious that a meaningful triaxial test on rocks whose uniaxial compressive strength may amount to several hundred MPa, would require such high lateral pressures as are found only in undisturbed rock masses at great depth but never where the rock has been involved in engineering or mining activities. The information required is the reaction of a rock when the naturally high lateral support has disappeared. The determination of the unconfined compressive strength of rocks in all stages of weathering, and occasionally also of soils, thus measures a natural property of geological materials which has also been used in the attempt to draw a distinctive line between rocks and soils (see Chapter 2).

Triaxial tests are used mainly in soil mechanics and a special modification was developed in 1946 in Texas (**McDowell**, 1954) as a method for road design. The uniaxial compression test is used particularly in rock mechanics and there are several versions for use in a laboratory or in the field (**International Society for Rock Mechanics**, 1972). In road engineering, the unconfined compressive strength is determined occasionally, especially for samples of stabilized bases, in which case a treated and not a natural material is tested.

Abrasion tests

There are a few more tests which could be used for the determination of the degree of disintegration and thus physical durability of natural road building materials. They are the abrasion tests, e.g. the Los Angeles Abrasion and the Deval Abrasion tests. In these tests, abrasion plays an important role, although, especially in the Los Angeles Abrasion Test, impact is almost as important as abrasion. The type of abrasion tested in these tests is basically different from that determined by the Accelerated Laboratory Polished Stone Value Test (BS 812).

In the Los Angeles Abrasion Test, the test sample, which consists of approximately 5 kg of graded, crushed or disintegrated rock, is contained in a revolving steel cylinder which has a steel shelf on the inner wall. This steel shelf lifts the sample in the revolving drum through about 90° before the sample falls back to the bottom and thus the shelf prevents the sample from simply gliding around the bottom of the drum. This lift, fall and temporary slip results of course in an alternation between impact when falling, and abrasion by mutual rubbing of the sample pieces during the other operations. The specifications for this test demand the addition of six to twelve steel balls of specified size and mass, the actual number depending on the grading of the sample. These steel balls which rotate with the sample, make the test an impact rather than an abrasion test. After completion of the test, which consists of a first run of 100 revolutions and, after rearrangement of the sample, a second run of 400 revolutions, the wear of the sample is determined as the percentage of the original sample passing the 4 750 micron sieve and retained on the 1 680 micron sieve.

The percentage retained on the 1 680 micron sieve varies considerably, depending on the type of rock. The decisive factors are amongst others the brittleness of the rock, its grain size, interlock of its mineral components, and degree of cementation. Quartzite, a strong but brittle rock, will normally give values between 15 and 20 per cent; granite, especially if medium- to coarse-grained, values which may be greater than 20 per cent; and basic crystalline rocks, which are often fine-grained with a strong intergrowth of the mineral components, values smaller than 10 per cent. Nevertheless, all the above rocks would make suitable surfacing or base aggregates.

As already mentioned, the Los Angeles Abrasion Test is an impact rather than an abrasion test. It has not been used frequently in Southern African road construction but the South African Railways use it extensively and it is an accepted means of determining aggregate quality in other parts of the world, e.g. in Australia, New Zealand and parts of the USA. The South African Railways specify that the abrasion of ballast stone should not be more than 22 per cent for main lines, not more than 28 per cent for branch lines, not more than 34 per cent for subsidiary, narrow-gauge lines and not more than 30 per cent for concrete aggregate subject to wear. These are rather strict limits which can be relaxed for road aggregates. In the United States, for instance, the limits vary between 40 and 50 per cent depending on the possible exposure of such aggregates to wear (**American Association of State Highway and Transportation Officials**, 1974).

Abrasion tests would probably be more revealing if no steel balls were used and even better if the drum did not contain the steel shelf. This is the case with the Deval Machine where the sample glides permanently on the bottom of a rotating, inclined drum. Mutual rubbing of the sample pieces and of the whole sample against the walls of the drum result in true abrasion. The information so obtained could be used as another expression of the strength of the material, particularly the strength of the bond between its mineral grains. The test could again be done on dry and wet material.

Although it appears more suitable for the determination of the resistance of rock to abrasion than the Los Angeles Abrasion Test, the Deval Machine has been used so seldom that no standards can be suggested for the interpretation of the test results.

DETERMINATION OF THE DEGREE OF DECOMPOSITION

Decomposition is caused by a chemical process and the use of chemical rock analyses for its determination and evaluation seems ideal. It must be realized, however, that chemical analyses yield information about the quantity of elements or,

in the case of rock analyses, about the quantity of oxides in terms of percentage mass. The quantity of these oxides hardly changes through the stages 'fresh', 'decomposed' and 'highly decomposed' except that some oxygen, hydrogen and perhaps carbon may be added. Noticeable changes in the percentage mass of the oxides, mainly through the loss of metals, do not occur until the decomposing rock reaches the stage of 'residual soil'. Chemical analyses therefore only indicate the changes through the stages of decomposition if considerably complicated recalculations of the analytical results are carried out.

Methods for such calculations are available. They are based mainly on the principles developed originally by **Cross** and others (1903, 1912). These methods are rather involved, however, and they require very accurate chemical analyses with exact determinations of particularly FeO, Fe_2O_3, H_2O^+, H_2O^- and CO_2 (see Chapter 5).

Before the decomposing rock changes into a residual soil, the principal chemical process is the mobilization and rearrangement of many of its constituents. This rearrangement of the elements leads to the transformation of most of the primary rock-forming minerals into such secondary ones which are in a better equilibrium with the surface conditions than the primary ones which developed under high temperature and pressure. Therefore, the type and particularly the quantity of secondary minerals present in a rock are an indication of the stage of decomposition the rock has attained. The more secondary minerals present, the more advanced is the decomposition and the more easily the remaining primary minerals will change into secondary ones.

The rate of change from primary to secondary minerals naturally affects the durability of natural road building materials which are obtained from sources such as igneous or certain metamorphic rocks. In their natural situation, these rocks have a stage of decomposition more or less in equilibrium with the environmental conditions and only, following for instance on surface erosion, when more moisture gains access to the rock will decomposition proceed further. When the rocks are removed from this situation, however, the equilibrium is disturbed and decomposition is re-established and continues. If such a material is placed in the layer of a pavement, this rate is likely to increase even further due to a moisture and temperature balance within the pavement, particularly in the base, which favours chemical activity. Care must therefore be taken that at the time of placing such a material decomposition has not proceeded so far that the further breakdown of the remaining primary minerals will cause the condition of the material to go below the standard required for the structural layer concerned. The continuation of decomposition cannot be avoided, but natural road building materials from decomposing sources can be selected so that there is sufficient play for the aggravated decomposition not to make the material fail. In certain borderline cases, by keeping the soil moisture alkaline (e.g. by stabilizing with lime or cement) continued decomposition can be retarded sufficiently to prevent the materials from failing before the expiry of the design life of the road (**Clauss**, 1967).

The determination of the percentage of secondary minerals

Since the development of secondary minerals from primary ones is the result of the decomposition of rocks which are able to undergo chemical weathering, a determination of the quantity of secondary minerals can replace a chemical rock analysis if information on the degree of decomposition is sought. In Appendix 1 a method is described which allows the determination of the percentage of secondary minerals in decomposing rocks by simple point counting from a microscopic slide. This method, which was developed by **D. B. van der Merwe** and which was first published in the National Institute for Road Research Bulletin No 5, 'Basic igneous rocks in road foundations' (**Weinert**, 1964), is designed so that the operator need not be familiar with any mineral which he sees under the microscope, although it would be an advantage if he could recognize quartz. The method is based solely on the very definite differences between the colour and brightness of primary and secondary minerals.

With the percentage of secondary minerals known, the position of a sample on the scale between completely fresh, 0 per cent secondary minerals, and completely decomposed, up to 100 per cent secondary minerals, can be determined. This provides two types of information with regard to the suitability of such a rock as a road building material:

a) The range of possible uses decreases as the percentage of secondary minerals increases. A rock which contains no secondary minerals could be used successfully in each structural layer in any type of road, although this is not done in practice because the use of a fresh rock for, for example, subbase or selected subgrade would be uneconomical, while a rock whose percentage of secondary minerals approaches 100 is useless in almost all cases.

b) More important than (a) is the possibility of using the percentage of secondary minerals to determine the durability of a decomposing rock when used in the structural layers of a pavement. Decomposition does not cease after the placing of such a material. It is therefore important to know what percentage of secondary minerals is acceptable in that it will provide sufficient play so that the unavoidable continuation of decomposition will not render the material unsuitable before the design life of the road expires, i.e. usually about 20 years. This limit depends on the layer concerned:

i) Only fresh rock is used as surfacing aggregate and, for the purpose of road construction, every rock which contains less than 15 per cent of secondary minerals can be regarded as 'fresh' under all environmental conditions (**Knight** and **Knight**, 1948; **Weinert**, 1964).

ii) The acceptable quality of decomposing rock for use in the *base* depends on the climatic environment, i.e. the N-value, and no percentage can therefore be fixed (Figure 11).

iii) A higher initial percentage of secondary minerals can be allowed in the aggregate for the *subbase* than in that for the base, but again the climatic environment must be considered (Figure 11).

iv) Durability problems of the kind described under (i) to (iii) do not exist for *selected subgrade* material. The consistency of this type of material is al-

ways that of a 'soil' in the colloquial sense – even if it is highly decomposed rock petrologically – and the criteria for selection need not take account of continued decomposition. Other criteria, particularly plasticity and bearing capacity, are paramount.

Materials intended for use in the *base* and *subbase*, therefore, must be selected most carefully because in these materials, if they are derived from decomposing crystalline rocks, the reaction of the initial condition of the material to the climatic environment becomes critical. This relation is shown in Figure 11 (see Chapter 6). In this diagram, the percentage of secondary minerals in a material intended for base or subbase is plotted relative to the N-value of the site concerned. The more to the left of the relevant durability line the plot is, the more durable the material is.

Summary

The method of determining the percentage of secondary minerals is very empiric and can be used by a person who knows practically nothing of geology, petrology or mineralogy. The purpose of this method is not the petrological description or classification of a rock but the determination of its stage of decomposition and, as a consequence, of the durability of a natural road building material under known conditions.

The result of the method described in Appendix 1 is that the following minerals are automatically counted as primary although their identity need not be determined:
quartz
feldspars
micas
amphiboles
pyroxenes
olivines
and in addition all accessories, for example the ores, such as magnetite, ilmenite and others, and minerals which occur mostly in metamorphic rocks, e.g. garnet, tourmaline and many more.

Most secondary minerals belong to the large family of clay minerals which are thus also determined automatically and again without being identified individually.

There are a few cases which require some special discussion:

Olivine often starts changing into *serpentine* before an olivine-rich ultra-basic rock has actually been exposed on the surface. After being exposed to surface conditions, olivine continues changing into serpentine and the serpentine eventually decomposes to become some type of clay mineral. The microscopic image of properly crystallized serpentine is that of a primary mineral and it must be counted as such. Progressing decomposition destroys the primary image and counting must follow this change.

Carbonate minerals, mostly in the form of the calcium carbonate *calcite*, are really only 'primary' in exceptional cases such as in carbonatite, a rare igneous rock which is of no importance in road construction, or in metamorphic rocks like marble. As soon as carbonates crystallize, they look like primary minerals under the microscope and they must be counted as such, but in the rare case when they are amorphous, they appear to be secondary and must then be counted accordingly. No new truly 'secondary' minerals develop from carbonate minerals which are only dissolved and reprecipitated as carbonates. The above procedure reflects, however, the potential effect which such minerals may have on the durability of natural road building materials.

The appearance of *volcanic glass* under a microscope is similar to that of isotropic minerals, i.e. it is dark under crossed nicols and colourless in ordinary light. Volcanic glass must be counted as a primary mineral. Since it is actually a gel, i.e. a liquid of very high viscosity, it changes into crystalline matter in time. The minerals which develop in this process are mostly very small and needle-shaped. These needles show up under the microscope like any other primary mineral, changing colour under crossed nicols. They may form a very dense network and problems may arise in recognizing the nature of the tiny areas of other matter between them. By using all the techniques described with the method for determining the percentage of secondary minerals, certainty about volcanic glass or the secondary nature of this matter must be obtained and counting done accordingly.

DESIGN TESTS

Once the durability of a natural road building material has been ascertained by means of one of the tests named or described so far, the material may be tested for design purposes. Although a number of tests have been suggested for the assessment of the physical durability of a natural road building material, only one of them need be used of course and the 10 per cent Fines Aggregate Crushing Test is particularly recommended.

The majority of tests which could be considered to be design tests have been established for such a long time, and special aspects of each of these tests have been discussed in so many papers, that such description will not be repeated here. These tests are standardized in various specifications, some of which are tabulated in Appendix 4.

These tests are intended to serve as a basis on which the road can be designed. This means that at this stage the material is not always tested in its natural condition but is often manipulated either by mixing several natural materials or by adding artificially produced ones such as lime, cement or bitumen, thus trying to simulate the condition in which the material will actually be used in the road. These tests can safely be carried out when the assurance has been obtained that the quality of the material is such that it will not change so much from its condition at the time of testing or construction that the design specifications will cease to be satisfied before the expiry of the design life of the road.

It is obvious that there is no clear-cut distinction between durability and design tests. This applies in particular to most of the tests which have been named as suitable for the assessment of the physical durability of natural road building ma-

terials. There are, on the other hand, design tests which can certainly not be used to determine the durability of a natural road building material. Such tests are especially those which involve thorough manipulation of the material or those where some artificially made material is added to the natural one.

In the design tests, the suitability of a material for a particular purpose, or the treatment of such a material to meet the demands of this purpose, are determined. This suitability for a particular purpose is not necessarily equivalent to durability and a durable material may still be unsuitable for the purpose in mind. Thus, quartzite is one of the most durable natural road building materials, yet it is occasionally unsuitable for use in road construction because its great strength makes recovery uneconomical, and it may be unsuitable for base and perhaps subbase because of its high pyrite contents (see Chapter 10) although this does not affect the durability of the stone.

These conditions of suitability or the measures to be taken to achieve the desired suitability are tested and determined in the 'design tests'. The number of relevant tests is large. They yield information on grading, moisture content, plasticity, bearing capacity, adhesion to binders, polishing and many other properties. They have been specified and have been in use for a long time. There is one test, however, which has not been specified but which may be a useful design test in certain cases.

INITIAL CONSUMPTION OF LIME (ICL)

There are natural road building materials which, in spite of being durable and satisfying the design requirements with respect to grading, plasticity and bearing capacity, fail because stabilization with lime or cement does not have the intended effect, particularly with respect to the gain of strength. Eades and Grim (1966) found that this was due to a number of chemical reactions, basically that of calcium with inorganic and in certain cases organic compounds contained in the road building material. Besides the absorption of calcium on the exchange sites of clays or the absorption of calcium in suitable unoccupied positions in decomposing minerals (which may be quick or long-lasting processes) direct reaction takes place with free silica, alumina and sulphates, the latter derived from the decomposition of pyrite which may be present in decomposing rocks and in a variety of soils. Most of these reactions are quite rapid and they are largely completed within something like an hour after the lime $(Ca(OH)_2)$ has been added to the material. During this process the pH of the soil moisture rises to 12,4 in most cases, which is the pH of the lime. It is obvious that the lime required for these reactions is used by the construction material and, in many cases, it will not then be available for the intended gain in strength. It should be realized, however, that the very high consumption of lime by amorphous silica (it may be more than 30 per cent), as well as other pozzolanic materials, does cause a strength gain which has been used, for instance, in the successful lime-stabilization of calcretes containing diatomaceous skeletons (Netterberg, 1971). Once the initial demand for lime for use in reactions like ion exchange, filling of unoccupied positions and the like is satisfied, the pH of 12,4 is attained and this indicates that lime is now available for the

desired purpose. In certain materials, particularly those which contain univalent ions, as e.g. Na$^+$, the pH will never reach 12,4; it will stay slightly below this value. In such a case, a constant lower pH indicates that the initial lime demand of the material has been satisfied.

Following the original investigations by Eades and Grim (1966), the test procedure was amended by Clauss to make the test more rigid and more directly applicable to road construction. The process determined by this test has been called the 'Initial Consumption of Lime' and the procudure has been described by Clauss and Loudon (1971). The test is carried out on material passing the 425 micron sieve and its details are given in Appendix 2.

Pure hydrated lime $(Ca(OH)_2,$ pro analysi) should be used during the selection of materials. In this way a standard can be created for the comparison of materials from different sites and in conjunction with service records. The quantity of lime consumed from the brand of road lime eventually selected for the job in hand, however, may not necessarily be precisely the same as that of the pure hydrated lime, and the test should therefore be repeated with the brand to be used. Account must also be taken of the slow reaction of lime, as distinct from cement; this means that, for some length of time, a certain quantity of calcium ions will remain available for slow processes of consumption, which are not necessarily all disadvantageous, and some lime may be lost by internal drainage.

The fact that the ICL test is carried out on material passing the 425 micron sieve requires some consideration. Since the grading of material used in the base or the subbase is such that the percentage of grains smaller than 425 microns is relatively small, the test result must be seen in relation to all the material in the layer concerned. This is necessary to avoid over-stabilization and excessive use of stabilizers. To make the results more realistic for practical application, the tendency has developed to test the material smaller than 2 mm which may comprise up to 50 per cent of all the material in a layer. Another advantage of the 2 mm grain-size is that virtually all grains larger than 2 mm, as well of course as many such grains smaller than 2 mm, are likely to be unweathered minerals or pieces of fresh rock which would not consume any significant quantity of lime.

In Rhodesia, materials as coarse as -4 750 microns is used for the determination of the initial consumption of lime, which means that the grain size specified in the South African method would account for only about 50 per cent of a sample tested in Rhodesia. In the Rhodesian sucrose method, the absorption of various quantities of lime by pure white cane sugar is measured by titration of a solution of calcium oxide and sugar in distilled water against N/10 hydrochloric acid, using phenolphthalein as an indicator. The quantity of titrant (in millilitres) required to make the phenolphthalein crystals show the red colour reaction is plotted as a standard curve which should always be a straight line as long as sufficient sugar has been added for a complete reaction. This curve is compared with a similar one obtained from the titration of the same mixture of sugar, lime and distilled water to which a specified quantity of the test soil, passing the 4 750 micron sieve, has been added. The two curves so obtained should run parallel, perhaps except for the portion referring to less than 2 per cent lime, and the one obtained from the mixture with soil should be below the standard curve. The distance between the two

curves, expressed in terms of the millilitre units of the ordinate (Figure 14), in relation to the slope of the curves is then used to calculate the initial consumption of lime. The details of this method have been published by the **Standard Association of Central Africa (1974). Clauss** and **Loudon** (1971) stated that an initial consumption of lime of less than 3,5 per cent by the -425 micron fraction of a natural road building material would not cause problems. As already discussed above, using a limit for a coarser sample may be misleading and a lower limit may have to be set. The Rhodesian method now uses material of such a grading that the -425 micron fraction will normally only account for about 50 per cent of the whole sample and, consequently, the limit should probably not be more than 2 per cent or possibly only 1,8 per cent. This would also be in agreement with the observation made in Rhodesia (**Rhodesian Government, Ministry of Roads and Road Transport,** 1972) that an initial consumption of lime of only 2,5 per cent apparently caused the failure of a runway prior to the expiry of its structural design life.
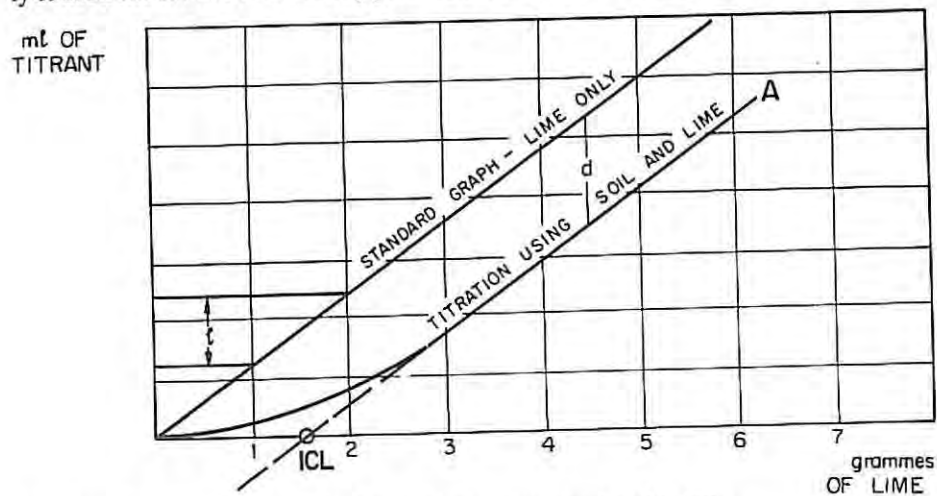
Figure 14: *Rhodesian sucrose method: Curves for the determination of the ICL*

FIELD INSPECTION OF NATURAL ROAD BUILDING MATERIALS

The distinction between the durability tests and design tests, and the recommendation that a material's durability should be assessed before it is tested further, may help to avoid unnecessary testing. In many design tests, for the sake of 'safety', more tests are carried out on a material than are really necessary. The appearance of a material and its feel between the fingers, for instance, can provide sufficient information about the possible plasticity of a weathered rock or a soil. The question may arise, therefore, whether a test of the plasticity of a natural material which turns out to be 'non-plastic' has been necessary at all. If an originally plastic material, e.g. a clayey soil, however, has been treated with lime and if the relevant test

thereafter indicates that it is 'non-plastic', the test has of course been necessary. Each test serves a particular purpose and yields information about a particular property of the tested material. Therefore, if the nature of a material makes it likely to lack a certain property and this lack can be confirmed easily, the execution of a test which determines this absent property will be a waste of time and effort.

During the prospecting and the field surveys for the location of suitable natural road building materials, an objective assessment of the stages of weathering and the potential suitability of a rock is advantageous. Absolute objectivity can of course only be obtained by exact laboratory investigations or tests. The method of evaluation described in Appendix 3 is based on the fourfold subdivision which has already been applied to the classification of the stages of weathering (Chapter 4) but the following method differs from this by its greater versatility.

When possible sources of materials are inspected, the conditions of a particular material are judged consciously or unconsciously by its colour and lustre, its hardness and consistency and its state of crystallization. Each of these three items is then subdivided into four special conditions and, after each has been determined visually according to its own merits and by applying simple arithmetic, an actually tenfold classification of stages of weathering results. The only equipment required for the method is a geological hammer, a hand-lens with a magnification of 8 or 10 and a steel needle or, if not available, a pocket knife.

This method is essentially only the systematization of what is done more or less unconsciously by each geologist or materials engineer when he inspects potentially suitable sources of natural road building materials in the field. This method, which was described previously in the National Institute for Road Research Bulletin No. 5 on 'Basic igneous rocks in road foundations' (**Weinert,** 1964), was designed primarily for the field assessment of the stage of weathering of crystalline rocks, i.e. of those rocks which, depending on the environmental conditions, can either disintegrate or decompose. With the necessary care, however, the method could also be used for rocks which only disintegrate, e.g. sandstone or shale, although there will seldom be a real need for this because hardness and consistency are actually the only properties which change during disintegration, while colour and lustre are hardly, and the state of crystallization never, affected.

The name 'Pick and click test' was once given to this method whose details are given in Appendix 3.

APPENDIX 1: *The determination of the percentage of secondary minerals*

The following method closely follows the one described by **D. B. van der Merwe** in the National Institute for Road Research Bulletin No 5: 'Basic igneous rocks in road foundations' (**Weinert,** 1964).

Sample preparation

Rocks are never entirely uniform entities and it is advisable, therefore, to prepare three slides from each condition of the rock as described below.

a) Fresh rock: Cut one or two slices of fresh rock about 3 mm thick from a larger selected piece with the aid of a diamond saw. Prepare thin sections as described below.

b) Weathered rock: If the material is hard enough, cut a few slices with a rock saw as explained in (a) above, but if the material is weak and disintegrates during sawing or grinding, impregnate it with a cohesive cement as described in (c) below.

c) Highly weathered rock: Generally this material is too brittle or too soft to be sawn and ground in the same way as pieces of fresh rock, and it must be treated with a suitable resin such as Lakeside 70C or with a mixture that includes Vestopal H and monostyrene among its ingredients.

There are many different resins on the market and new ones are constantly being developed. Each has its advantages and disadvantages. The use of some of them involves relatively high temperatures which may have some effect on hydrated minerals, especially clay, but others react at room temperature, e.g. the very liquid EPOFIX. The choice of the resin for the preparation of microscopic slides of brittle, soft or very porous materials must be guided mainly by four requirements:

a) the resin must be sufficiently liquid at the time of application to penetrate a soil-like material like a highly decomposed rock;

b) after hardening, the resin must be strong enough to stand the process of grinding and polishing;

c) the resin must not melt or otherwise change when the sample is stuck on the glass slide; and

d) the refractive index of the resin must be as close to that of glass as possible.

The two methods described below were previously described in the National Institute for Road Research Bulletin No. 5 'Basic igneous rocks in road construction' (**Weinert**, 1964). They are given again here because they have worked satisfactorily in the past, although it is not claimed that they are the only possible methods (**Fowler** and **Shirley**, 1967).

i) Method using Lakeside 70C cement:

Select an undisturbed, highly weathered sample which must be shaped into a block of about 20 x 20 x 10 mm. Heat the sample thoroughly for about half an hour on a hot plate with a surface temperature of 180-200 °C, or hold it in a Bunsen flame for about 15 minutes. When it is heated and dried, drop the sample quickly into a vessel containing xylene. The sudden immersion causes immediate effervescence as the xylene replaces the air in the sample. On another hot plate with a surface temperature of about 180 °C heat up the following mixture to boiling point: one part by mass crushed Lakeside 70C cement in eight parts by mass 96 per cent ethyl alcohol.

TABLE 12
Initial hardening mixture

Ingredient	Hardening time	
	6 days	1 month
Vestopal H	50 cm³	75 cm³
Monostyrene	42 cm³	60 cm³
Cobalt Octoate	0,3 cm³	0,1 cm³
Cyclonox	0,6 cm³	0,2 cm³

TABLE 13
Final hardening mixture

Ingredient	Hardening time	
	6 days	1 month
Vestopal H	50 cm³	50 cm³
Cobalt Octoate	0,2 cm³	0,1 cm³
Cyclonox	0,2 cm³	0,1 cm³

Now remove the sample from the xylene and place it in the Lakeside mixture. Allow to boil gently until all the spirits have evaporated, and the cement is in a molten condition. If this condition is correct, a little bit of the cement drawn out with a glass rod becomes brittle when it is cooled rapidly in air. Remove the sample from the mixture and allow it to cool to room temperature. The samples can now be ground like any other rock.

ii) Method using a Vestopal H and Monostyrene mixture:

The hardening mixture contains the following chemicals:

1) Vestopal H (a resin)
2) Monostyrene (thinner)
3) Cobalt Octoate (1% Co)
4) Cyclonox LT 50

Select an undisturbed sample and trim to form a block of 20 x 20 x 10 mm. Allow to dry at room temperature because too much moisture in the sample will affect thorough penetration by the resin.

Mix the four chemicals before adding them to the sample. By altering the quantities of the chemicals, the hardening time can be lengthened or shortened. Thorough penetration of clays may take quite a long time, up to about one month, and an adjustment of the mixture according to permeability is, therefore, required. Porous materials, such as highly weathered igneous or metamorphic rocks, but also sandstone, can be hardened rapidly but a much slower hardening process is necessary for heavy clays. Mixtures suitable for hardening both types are given in Table 12.

Place the sample in a container and add the misture. The top of the sample must be left just uncovered. Place the container in a desiccator which is then evacuated. Add more hardening mixture while penetration is in progress. When no further penetration of the mixture can be observed, the following mixture (Table 13) without the thinner monostyrene can be added to ensure complete penetration by Vestopal H, the actual hardening agent.

Once hardening is complete, the samples must be ground as described in the next section. A thin oil instead of water is recommended as a lubricant. The main advantage of the second method is that impregnation and polymerization take place at room temperature, thus preventing possible changes in secondary minerals through excessive heat. The disadvantage of this method is the long period required for hardening which in the first method, using Lakeside 70C only, takes about one hour.

d) *Soil*: The soil can be either residual or transported or a mixture of both. If an undisturbed sample is available, harden it as described in (c) above and if not, hardening must be carried out with Epicote cement as follows:

Thoroughly mix about 20 g Epicote 815 with 4 to 5 g Diethylene-Triamine (DTA), the hardening agent. Stir well. Pulverize about 30 g of the material to pass a 4,76 mm sieve onto a piece of hard paper. Mix the pulverized soil thoroughly with the Epicote mixture and leave for about 12 hours to harden. Grind to a slice and prepare the slice as indicated below.

Preparation of microscopic slides
Having prepared the sample by one of the methods described, proceed as follows:

i) On a wet square of plate glass or on a wet grinding wheel, grind one surface of the slice, starting with grain-size 220 silicon carbide abrasive powder and using successively finer abrasives. Wash the slice carefully after each stage of grinding. Finish grinding with No. 95 carborundum aloxite abrasive using another grinding plate as before. The final surface of the slice must be clean and free of any grooves and holes.

ii) Place the rock slice with smooth face upwards, and a glass slide (25 x 75 mm) on a hot plate with a surface temperature of 150 °C ($\pm$ 5 °C).

Spread a thin layer of Lakeside 70C or liquid Canada Balsam (preferably Lakeside 70C) over the smooth surface of the rock slice as well as over the glass sslide. If Canada Balsam is used, allow it to evaporate until a thin thread drawn out of the balsam with a needle becomes brittle when it is cooled rapidly. Remove the rock slice and quickly press the coated surfaces together firmly so that *bubbles do not occur in the mounting cement.*

If the sample has already been hardened by the method described in (c) or (d) above, heat only the glass slide because the hardening cement in the pores of the sample will melt on heating.

iii) Reduce the thickness of the slide on a grinding wheel until coloured minerals become noticeably translucent. Start with No. 220 silicon carbide powder and use succesively finer abrasives until slice and mounting cement have a thickness as shown below. Finish the grinding by hand with a uniform rotary motion on a clean square of plate glass on a rust-free metal block, using No. 95 carborundum abrasive. Measure the thickness of the slice and mounting cement with an ordinary micrometer. If the cement is reasonably thin, the following thicknesses will make the sample suitable for microscopic investigation:

Fresh rock: 0,06 mm
Weathered rock: 0,10 mm
Highly weathered rock: 0,10 to 0,15 mm
Soil: 0,10 to 0,15 mm

Counting the primary and secondary minerals
General
a) A slide is a slice of rock stuck on a rectangular piece of glass with a resin, and ground down to a thickness of about 0,03 mm (0,06 mm including thickness of mounting cement).

b) When observed under a microscope at a magnification of about 60, the rock slice appears as a mosaic of many minerals. Under ordinary light, the minerals are mostly colourless and transparent or occasionally slightly tinted or light brown. When viewed between crossed nicols, however, they take on a variety of colours.

c) Make a total of a little more than 500 counts and calculate the percentages of primary and secondary minerals. It has been found experimentally that a count of little more than 500 is accurate enough but the rows in which to count must be spaced evenly so as to cover the whole slide. Make and count at least two, or even better three, slides from each sample. A trained operator can complete the 500 counts in about 15 minutes.

Counting
a) Count the primary and secondary minerals in each slide and record the number of primary and secondary minerals separately.

b) Count while the slide is viewed between crossed nicols. *Ordinary light and filters must also be used for the easier recognition of minerals.*

c) Count the mineral directly under the cross-hairs in the eye piece of the microscope. Move the slide on automatically by hand and count the next mineral falling under the cross-hairs. Ignore the minerals passed while the slide is moving. If the mineral which has just been counted is of such a size that it falls under the cross-hairs again, *it must be counted again.*

Recognition of primary and secondary minerals
i) *Primary minerals* observed between crossed nicols nearly always have sharp borders and bright or clear colours. They show also very distinct colour changes and become lighter or darker when the microscope stage is rotated through at least 90°. One of the colour changes is always towards black or at least dark grey. When observed in ordinary light, the primary minerals are colourless or faintly coloured and almost transparent.

b) *Ore* minerals are also *primary minerals* and they are counted accordingly. They appear black with sharp borders and show no colour change on rotation. *When viewed in ordinary light, they remain black.*

c) *Isotropic* minerals are *primary*. They are dark between crossed nicols with no colour change on rotation of the stage. They are completely *colourless in ordinary light* and must be counted as *primary*.

ii) *Secondary minerals* are those which are usually brown or brownish in colour and which do not change colour on rotation between crossed nicols. When seen in ordinary light they are still brownish and very distinct from the colourless primary minerals (*Plates* 20, 21, 22 and 23).

Special cases not to be counted
When the cross-hairs fall on any of the conditions described below, the point must not be counted and the slide must be moved on.
a) Cracks in the rock slides are usually filled with resin which can give the appearance of an ore mineral or a secondary mineral when observed between cros-

sed nicols. When the crack is viewed in ordinary light, however, it is completely colourless and *bright* red when viewed through a gypsum filter.

b) Where the grinding of the slide has caused the removal of a mineral, the gap left has the same appearance as the crack in (a) above, and the same procedure is followed for identification.

c) Where the edges of a primary and a secondary mineral overlap, a dull zone is formed as shown in Figure 15. If the cross-hairs fall directly on such point, do not count it!
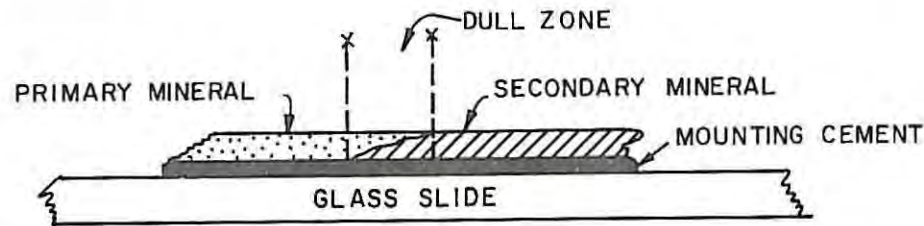
Figure 15: *Section through overlapping primary and secondary minerals*

Special cases to be counted

a) A secondary mineral may include many relatively bright spots which are relics of primary minerals. If the cross-hairs fall on such a bright spot, it must be counted as primary and if they fall on the brownish part, as secondary (see Figure 16).
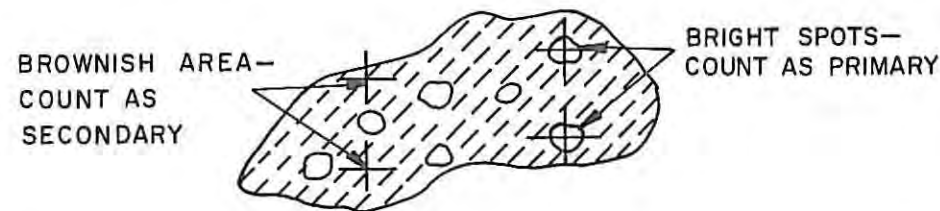
Figure 16: *Large secondary minerals with relics of primary minerals and different positions of cross-hairs of microscope*

b) Certain minerals, e.g. biotite, may appear to be secondary minerals when viewed between crossed nicols, as well as in direct light, because they may be rath-

er dark and show little colour change on rotation. Care must be taken, therefore, *to count as primary* those minerals which also show only *slight colour changes between darker and lighter* on rotation of the stage between crossed nicols, i.e. the pleochroic minerals.

c) Where the edges of two primary minerals overlap, the overlap may appear dark, but when viewed in ordinary light it will be colourless and *must* be counted as a primary mineral.

General remarks

a) The microscope stage holding the slide must be frequently rotated through a few degrees to enable rapid recognition of the primary minerals by colour changes.

b) In doubtful cases, it is helpful to view the slide frequently in ordinary light and through a gypsum filter, especially for the detection of cracks and gaps and also for the recognition of ores and isotropic minerals.

c) Before starting to count, observe the slide through a smaller magnification than 60 to obtain an overall picture.

d) To get used to the procedure, for the first time at least, three or four slides must be counted simply for practice purposes.

Equipment and chemicals required

1) Rock saw for cutting sections from hard rock samples.
2) Special hot-plate, with thermostat, for heating glass slides and samples before mounting.
3) Rotating grinding wheels (metal) for grinding rock slides. These can be bought or they can be constructed in a workshop.
4) The following grinding powders are required for grinding slides:
No. C220 silicon carbide grain grit
No. C320 silicon carbide grain grit
No. C400 silicon carbide grain grit
No. C600 silicon carbide grain grit
No. 95 optical smoothing powder
5) Double hot-plate for heating rock slices and hardening mixtures.
6) Lakeside 70C, a chemical cement, which is provided in about pencil-thick bars. Other chemical cements may also be used if they comply with the requirements given above (**Fowler** and **Shirley**, 1967).
7) Chemicals:
   i) Vestopal H
   ii) Monostyrene
   iii) Cobalt Octoate (1% Co)
   iv) Cyclonox LT 50
8) Epicote 815 and Diethylene-Triamine (DTA)
9) Micrometer for measuring the thickness of slides.
10) Ordinary polarizing microscope, student model with rotating stage. It has been found that a binocular stereomicroscope helps to reduce eye strain considerably during counting.
The handling of a microscope can be demonstrated by any microscope salesman

or may be read up in a good handbook on optical mineralogy, but the following precautions are important:

a) Never use the microscope in a dusty room.
b) Never use or leave the microscope in direct sunlight.
c) Close the microscope up in its cupboard after use.
d) Use a soft cloth or camel-hair brush to clean the lenses and use benzol, never alcohol, for cleaning the microscope.
e) The objective lens must *never touch the slide*: in focusing on a large object always start from the lowest possible position.

11) Electrical or mechanical counter: an electrical counter permits faster and more accurate counting. The electrical stage, being part of the counter, automatically shifts the slide onwards and replaces manual shifting. Electrical and mechanical counters can be bought or a simple mechanical counter can be constructed in a workshop. With the use of such counters, it is not necessary to remove the eyes from the ocular after every count.

APPENDIX 2: *Determination of the initial consumption of lime*

The South African method for the determination of the initial consumption of lime (ICL) as designed by **Clauss** and **Loudon** (1971), based on the method originally developed by **Eades** and **Grim** (1966), is given below.

Test procedure: The test for the determination of the initial consumption of lime (ICL) differs from the original test procedure by the following modifications.

a) The use of a standardized lime ($Ca(OH)_2$, *pro analysi*) test in addition to a road-lime test in order to obtain a rigid definition of the initial consumption of lime of a material, and so make possible a comparison of materials from different localities.

b) The use of a correction factor, worked out for the ICL of the water used in the test, which increases the accuracy of the determination considerably.

The following *equipment* is required:

Balance (accuracy: 0,01 g)
pH meter (accuracy: 0,02 units)
5 plastic bottles, screw-top (150 ml)
Spatula
Soft tissues
Glass beaker (200 ml)
Distilled water
Calcium hydroxide (*pro analysi*)
Wash-bottle (for cleaning electrode of pH meter)
Correction tables

The corrected *results are recorded* to the nearest 0,5 per cent lime required to saturate the soil/water mixture as indicated by the pH of the slurry.

The actual *testing* is done in nine steps:

i) Representative samples of air-dried material, passing the 425 micron sieve, to equal 20 g of oven-dried material, are weighed to the nearest 0,1 g and poured into 150 ml (or larger) plastic bottles with screw tops.

ii) Since most materials require 2 to 5 per cent lime, it is advisable to set up five bottles with lime percentages of 2, 3, 4, 5 and 6. This will ensure, in most cases, that the percentage of lime required can be determined in one hour. Weigh the lime to the nearest 0,01 g and add it to the soil. Shake to mix the soil and dry lime.

iii) Add 100 ml of CO_2-free distilled water to the bottles.

iv) Shake the soil-lime and water until there is no evidence of dry material on the bottom. Shake for a minimum of 30 seconds.

v) Shake the bottles for 30 seconds every ten minutes.

vi) After one hour, transfer part of the slurry to a plastic beaker and measure the pH of the slurry. The pH meter should preferably be equipped with an electrode specially for high pH, calibrated with a buffer solution of pH 12.

vii) Record the pH of each of the soil/lime-water mixtures. The lowest percentage of lime at which the pH remains constant (in most cases at pH = 12,4) is the percentage to saturate the soil/water mixture.

viii) Determine the percentage of lime required for the saturation of the water only (100 ml).

ix) The ICL of the samples is determined by correcting the percentage of lime, obtained in step (vii), for saturation of the water only. This is done by reference to Figure 17.

APPENDIX 3: *The 'pick and click test' for the determination of the degree of weathering*

The following test should be done by two persons. The one collects the sample, does the test in the quarry or outcrop, and studies the material in its natural environment and in relation to the surrounding rock; the other inspects the sample without knowledge of the sampling site. If the test cannot be done in the quarry, it should be carried out independently by two persons in the laboratory. The results obtained should show little variation but borderline cases may occur which are clarified better after two independent inspections.

Three properties of the sample must be inspected. They are:

a) colour and lustre
b) hardness and consistency
c) state of crystallization

No special geological or mineralogical knowledge is required. The investigator must know, however, what is a rock and what is a soil, that rocks are composed of minerals which occur as crystals and that a weathered rock looks quite different from a fresh one. He must also be able to recognize quartz with certainty.

Each of the above properties (a), (b) and (c) must be inspected independently. One of the values 1, 2, 3 or 4 is given to each property according to the description given below. The three values so obtained for any sample are then summed and the result will be a figure between 3 and 12. This figure classifies the sample into one of the four conditions 'fresh', 'weathered', 'highly weathered' or 'residual soil'.

% LIME REQUIRED FOR SATURATION OF WATER ADDED TO SAMPLE (Z)

I.C.L OF SAMPLE ONLY (%)

% LIME ADDED TO SAMPLE (X)
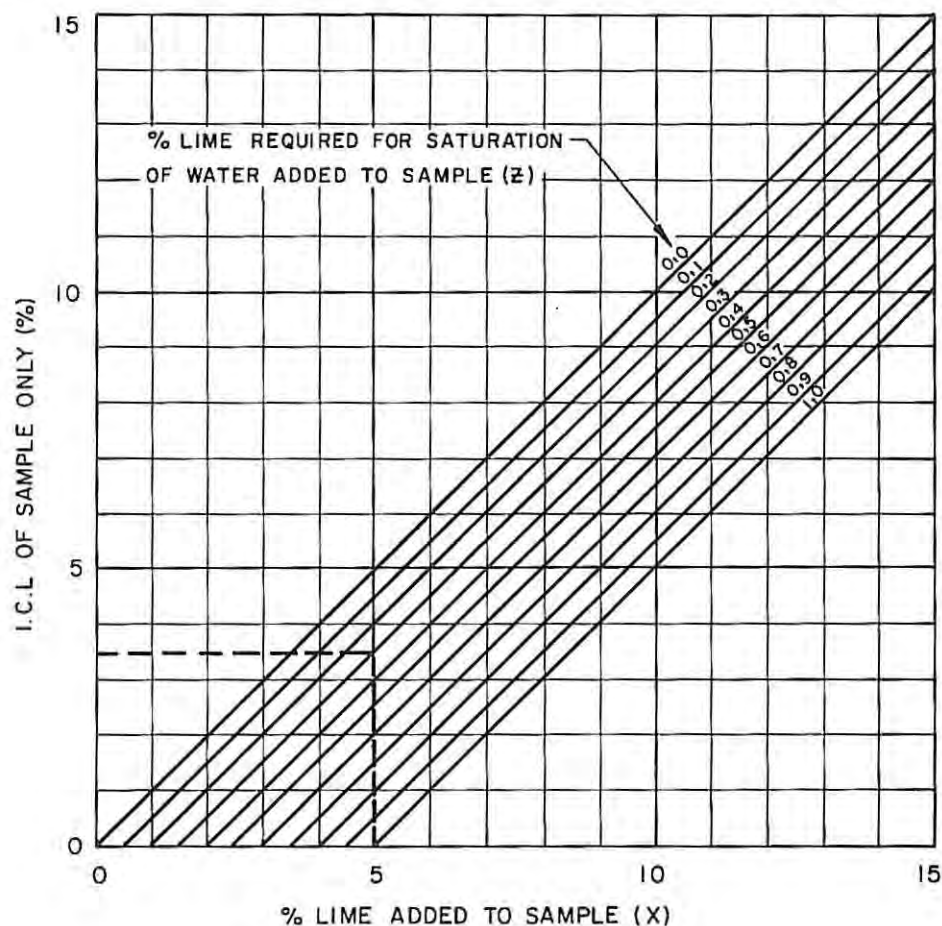
Figure 17: *ICL correction curves for saturation of water (after Clauss and Loudon, 1971)*

$I.C.L. = X - 5Z$ *Where X = % Lime added to sample*
Z = *% Lime required for saturation of distilled water added to the dry sample.*

Test procedure

The following instructions must be read carefully before starting determination.

a) Do not judge rock pieces lying loosely about in a quarry or outcrop.

b) Always inspect a newly crushed surface of the rock.

c) Make sure that loose material on the sample is *not* surface soil sticking to it.

d) Take *everything* mentioned in the description under each index value into account and do not judge simply on the apparently most convenient point.

e) Do not observe in direct sunlight.

f) An increasing index value in the description of each of the three properties indicates an increasing degree of weathering in regard to this particular property.

Index Value	Description of condition

Colour and lustre

(Do not use hand-lens, decide from general impression. Remark 'b' above refers in particular.)

1 Shiny, flickers when turned; dense cementing matrix, if present, looks shiny; if present, iron oxide appears only as film on fissures.

2 Lustre generally dull; natural colour and lustre of fresh rock partly retained; light-coloured crystals may still reflect light but dark ones will not; dense cementing matrix, if present, looks dull.

3 Dull yellowish; dull light grey; dull light brownish.

4 Earthy red; earthy black when wet or earthy grey when dry; whitish colours may also occur.

Hardness and consistency

(Judge the whole sample and not the surface only.)

1 Hard; rings when struck with a hammer; breaks into sharp chips. Blasting required.

2 Still hard but breaks along natural cracks (often with flat surfaces) which are strongly stained with iron oxide; fresh cores surrounded by a clearly visible layer of weathered material. Picked with difficulty; blasting required in places.

3 Breaks easily into small pieces ('sugar') many of which are loose crystals; friable when rubbed slightly with finger; crystals and rock pieces break off when rubbed slightly with finger. Picked easily.

4 Soft; earthy; gravelly with diameter of grains less than 10 mm. Shovelled easily.

State of crystallization

(Use hand-lens; use needle to determine brittleness or softness of crystals. Remark 'b' above refers in particular.)

1 Crystals well defined with sharp boundaries between them; up to 10 per cent of the crystals may be oxidized, brittle or powdery; dense cementing matrix, if present, contains excellently shaped crystals. If there are no scattered crystals

in the dense cementing matrix the above comments cannot be applied and the sample should be allotted the same value that was given for 'colour and lustre'. (This is a rare case where 'colour and lustre' must be judged twice.)

2   More than 50 per cent of all crystals are as well defined as under 1 above; more than 10 per cent and up to 50 per cent of the crystals are oxidized, brittle or powdery; apply the same rules given under 1 above to dense cementing matrix with or without loose crystals.

3   More than 50 per cent of the crystals are oxidized or powdery. Their original shape, however, may be largely retained, i.e. light-coloured crystals that are present stand out clearly.

4   No crystals, except quartz, detectable.

Give an index value to each of the properties – *colour and lustre, hardness and consistency, state of crystallization* – and add the three numbers. The sum obtained classifies the sample as follows:

TABLE 14
Classification of weathering according to weathering values

Sum of index values	Classification
3, 4	Fresh
5, 6, 7	Weathered
8, 9, 10	Highly weathered
11, 12	Residual soil

Complete the inspection and classification of one sample *before* starting on the next one.

The above determination can be applied to any type of crystalline rock although the method was originally designed for basic igneous rocks only. It offers little advantage, however, if applied to other types of rock.

The interpretation of the results offers several possibilities:

A higher value for hardness and consistency than for the state of crystallization indicates predominance of disintegration. If both values are equal, a lower value for lustre indicates a slight predominance of disintegration over decomposition, while decomposition is more prominent in all other cases.

The values 1 and 2, and in exceptional cases also that of 3, for *hardness and consistency* make the material suitable for crushing tests. The 10% Fines Aggregate Crushing Test is again recommended particularly because of its special suitability for strong as well as weak materials.

The index values given to *state of crystallization* indicate very roughly the percentage of secondary minerals in the materials:

A value '1' for state of crystallization indicates less than 15 per cent secondary minerals present. If lustre is also '1', this percentage will be lower than when lustre is '2'.

A value of '2' for state of crystallization indicates 15 to 30 per cent of secondary minerals. A higher value for hardness and consistency than '2' indicates a percentage of secondary minerals near the lower end of the range. '2' or even '1' for hardness and consistency, combined with state of crystallization '2', indicates a percent-

DISINTEGRATION

COLOUR AND LUSTRE + HARDNESS AND CONSISTENCY

DECOMPOSITION

COLOUR AND LUSTRE + STATE OF CRYSTALLIZATION

	2	3	4	5	6	7	8
2	C	C	S	S	X	X	X
3	C	C/S	S	S	S	X	X
4	X	I	S	S	S	X	X
5	X	X	I	I	I	S	X
6	X	X	X	I	I	I	X
7	X	X	X	X	U	U	U
8	X	X	X	X	X	U	U

Condition :

C   Suitable for chips and crushed stone.

C/S   Suitable for chips and crushed stone if "state of crystallization" is "1"; otherwise treat like S.

S   Suitable base material under all environmental conditions.

I   Intermediate : Suspect base material if N<5, generally suitable subbase material.

U   Suspect subbase material under all environmental conditions, may be suitable for selected subgrade if "state of crystallization" is not '4'.

X   Cannot exist, repeat determination.

Figure 18: *Diagram for the interpretation of the weathering value*

age of secondary minerals in the middle or upper half of the range. Most combinations in this range will be 2-2-2.

A value of '3' for state of crystallization indicates more than 30 per cent of secondary minerals. This percentage may be below 30 when hardness and consistency are determined at '4' because of the high degree of staining with iron oxide of an otherwise rather fresh, but highly disintegrated, material. The result of the determination will be 3-3-3 in the majority of cases.

A value of '4' for state of crystallization indicates a very high percentage of secondary minerals, more than 50 per cent in most cases; a definite lower limit, however, cannot be given.

Atterberg Limits are only of use when the value for hardness and consistency and that for state of crystallization is '3' for the one and '4' for the other or when both properties are '4'. The determination of these Limits is of little use when both properties are determined as '3' or less. This restricts the applicability of the Atterberg Limits to the actual residual soil and the most extreme forms of highly decomposed rock which must be treated in the same way as soils.

Decomposition produces aggregates which are more frequently unstable than are the products of disintegration. The determination of the degree of decomposition or disintegration which has occurred in a weathered rock is, therefore, of vital importance and Figure 18 serves as a guide to interpreting the numerical values assigned to the properties of rocks which have undergone either of these two forms of weathering.

Figure 18 indicates the degree of decomposition on the ordinate and the degree of disintegration on the abscissa. The degree of *decomposition* is expressed by the sum of the values given for

colour and lustre + state of crystallization

and the degree of *disintegration* by the sum of the values given for

colour and lustre + hardness and consistency.

The values 2, 3, 4, 5, 6, 7 and 8 are thus obtained on both co-ordinates. *Colour and lustre* is common to both co-ordinates because any alteration in a rock will have some effect on its lustre, but this effect will be much more pronounced if decomposition predominates.

The number of squares thus obtained is 49. Twenty-four of these squares, however, cannot occur in practice because of their contradicting value combination, e.g. lustre = 1 cannot occur with state of crystallization = 3 or 4 although it could well occur with any value for hardness and consistency. Eight of the remaining 25 squares are defined by two possible combinations and the remaining 17 by one only.

Many results will fall into equally indexed fields, i.e. 2/2, 3/3, etc., which represent the progress of weathering where decomposition is associated with a corresponding degree of disintegration. The squares above this mean line are the area of disintegration and those below, the area of decomposition. The greater number of possible variations in the area of disintegration is caused by the fact that a rock can break down physically almost completely without important chemical alteration of its constituents while, when decomposition predominates, weathering will always be associated with a certain degree of disintegration.

There are five major groups of squares each with a distinguishing letter; the letter indicates certain properties of a material and is a guide to the interpretation of the test results:

C, suitable for surfacing chips and crushed stone. This is all fresh rock and such material is normally found in quarries cut into the slope of a hill. In flat areas where N<5 deep quarrying may be necessary to obtain such material.

C/S. This is an intermediate group between the groups C above and S below. This material is also suitable for chips and crushed stones when decomposition is unimportant, i.e. when the value of the state of crystallization is 1. If the state of crystallization is determined as 2, however, this material should be treated in the same way as the next group, S.

S, suitable for natural gravel bases under all environmental conditions. The ideal material would fall into the squares: Disintegration = 6, Decomposition = 4 or 7/5, where the rock has disintegrated completely as a gravel, and where the decomposition (state of crystallization having a value of 2 in both cases) has not yet reached such a point as to make the material unstable within the lifetime of the road.

I. This is again an intermediate state, this time between S above and U below, indicating suspect base material when N<5 although the material may be considered for subbase. No difficulties should be expected with such material in dry regions (N>5). Attention is drawn to the following: The line N = 5 is *not* a completely sharp border line where a material one kilometre east of it will perform completely differently from a similar material one kilometre west of it. It must be understood that the line N = 5 marks a narrow border zone of about 20 to 30 km wide.

U. Any weathered crystalline rock must be regarded as unsuitable for subbase under all environmental conditions but when the 'state of crystallization' is not determined as '4', it may be considered for selected subgrade or fill.

x. Determinations which fall into these areas must be repeated because some of the determinations will be contradictory, i.e. describing conditions which cannot occur simultaneously.

APPENDIX 4: *Test specifications*

The following tables record references to test specifications which are intended to yield information on properties of natural road construction materials. Tests in which such materials are used only as part of the test specimen without aiming at the determination of the properties of the natural material, e.g. the so called Marshall test, are not included in these tables.

The names and details of tests that generally aim at the determination of the same or very similar properties of materials vary. None of the different names of similar tests have therefore been used in the tables. The tests have been arranged according to the property measured, and the names generally used by Southern African engineers for the property concerned, are used. These names have been arranged alphabetically and, since there are occasionally several colloquial names for a test, certain tests are given more than once in the tables.

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Property determined by test	Technical Methods for Highways (TMH) No. 1	South African Bureau of Standards (SABS) Method No.:	Standards Association of Central Africa (CAS)	British Standards Specification (BS)	AASHTO (formerly AASHO)	ASTM	Californian Standard	Remarks
Abrasion – Deval machine					T4 - 35	D2 - 33	210 C	
Los Angeles machine		846	A43/2 nos. 14 & 15		T96 - 45	C131-64T	211 D	
Accelerated laboratory polished-stone value		848		812				
Adhesion test (bitumen – stone)	B11							Based on West German test: Riedel and Weber test
Aggregate Crushing Value (ACV)	B1	841		812				
Aggregate Impact Tests				812				See 'Treton test'
Aggregate Soundness Tests								See 'Freezing-Thawing', 'Magnesium sulphate' and 'Sodium sulphate' tests.
Apparent bulk specific gravity	B14			812	T85 - 60 T84	C127-68 D854-58	206 D 207 D 208 B	
Atterberg Limits								See 'Linear shrinkage', 'Liquid Limit' and 'Plasticity Index'
Average Least Dimension	B18							
British Aggregate Impact Test				812				
Bulk density				812				See 'Density'

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Property determined by test	Technical Methods for Highways (TMH) No. 1	South African Bureau of Standards (SABS) Method no.:	Standards Association of Central Africa (CAS)	British Standards Specification (BS)	AASHTO (formerly AASHO)	ASTM	Californian Standard	Remarks
Bulk specific gravity	B14			812	T85 T85-60	C127-68 D854-58	206 D 207 D 208 B	
California Bearing Ratio (CBR)	A8		A43/2 nos. 3 & 5	1377	T193-63	D1883-67		
Compressive strength triaxial			A43/2 no. 8	1377				
unconfined	A14		A43/2 no. 11	1377				
Density	A7	864 865	A43/1 no. 21	1377	T180-61	D1557-64T		
Deval Machine (abrasion)					T4-35	D2-33	210 C	
Dust content of road aggregate	B5			812	T11-60	D1140-71		
10% Fines Aggregate Crushing Test (10% FACT)	B2	842	A43/2 nos. 12, 13	812				
Fines (plastic) in aggregate					T210-64			
Flakiness Index		847		812				
Freezing-Thawing Test					T103		528	

TEST SPECIFICATION

Property determined by test	Technical Methods for Highways (TMH) No. 1	South African Bureau of Standards (SABS) Method No.:	Standards Association of Central Africa (CAS)	British Standards Specification (BS)	AASHTO (formerly AASHO)	ASTM	Californian Standard	Remarks
Grading Analysis	B4	829	A43/1 no. 5	812	T27-60	D422-72	202 G	
Hardness (of aggregate)					T189-63	C235-62T		
Impact Value (Treton)	B7							
Impact Test (British)				812				
Initial Consumption of Lime (ICL)			A43/2 no. 18					This is the sucrose method on the –4 750 mm fraction used in Rhodesia
Least dimension (of aggregate)	B18							
Linear shrinkage	A4	853	A43/1 no. 11	1377	T92-60	D427-90	228 A	
Liquid limit	A2	851	A43/1 no. 8	1377	T89-60	D432-54T		
Los Angeles Abrasion		846	A43/2 nos. 14, 15		T96-45	C131-64T	211 D	
Magnesium sulphate soundness		839			T 104		214 D	
Particle index of aggregate								'Geometric characteristics of aggregate' by Huang, University of Leeds
pH of soil	A20	854		1377				
Plastic fines in aggregate					T210-64			

TEST SPECIFICATIONS

Property determined by test	Technical Methods for Highways (TMH) No. 1	South African Bureau of Standards (SABS) Method No.:	Standards Association of Central Africa (CAS)	British Standards Specification (BS)	AASHTO (formerly AASHO)	ASTM	Californian Standard	Remarks
Plasticity Index	A3	852	A43/2 nos. 9, 10	1377	T90-61	D424-54T		
Polished-stone Value (PSV)		848		812				
Sand equivalent		838	A43/1 no. 21	1377	T210-64	D2419-69	217 I	
Scratch hardness of coarse aggregate particles					T189-63	C235-62T		
Sieve analysis								See 'Grading analysis'
Sodium sulphate soundness·					T 104		214 D	
Soluble salts	B16	849						See also 'Soluble sulphates'
Soluble sulphates	B17	850		1377				See also 'Soluble salts'
Soundness tests								See 'Freezing-Thawing', 'Magnesium sulphate' and 'Sodium sulphate' tests
Specific gravity of soil	B14 B15	844	A43/1 nos. 12, 13	1377	D854-58			
Standard size of coarse aggregate for highway construction					D854-58			
'Sunburn' of aggregate								German test: DIN 52106 (see Chapter 10)
Treton impact test	B7							

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TEST SPECIFICATIONS

	Triaxial Compression Test	Unconfined Compression Test	Uniaxial Compression Test	Water absorption	Weathering tests
Remarks	This includes the Texas triaxial test		See 'Unconfined Compression Test'		See 'Soundness tests'
Californian Standard				206 D 207 D	
ASTM				C 127 D 854	
AASHTO (formerly AASHO)				T 85	
British Standards Specification (BS)	1377	1377		812	
Standards Association of Central Africa (CAS)	A43/2 no. 8	A43/2 no. 11			
South African Bureau of Standards (SABS) Method No.:				843	
Technical Methods for Highways (TMH) No. 1		A 14		B14 B15	
Property determined by test	Triaxial Compression Test	Unconfined Compression Test	Uniaxial Compression Test	Water absorption	Weathering tests

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CHAPTER 8
CLASSIFICATION OF NATURAL ROAD CONSTRUCTION MATERIALS

THE PROBLEMS OF A CLASSIFICATION

If it is stated that 'about 40 types of rock' are used for road construction in Southern Africa, then 'about' has been added to the figure 40 deliberately. Rocks are not such strictly defined entities that a granite is always a granite beyond doubt, but there are continuous transitions from granite to syenite, diorite, pegmatite, quartz-porphyry or orthogneiss, from shale to slate, sandstone or hornfels, and each rock may undergo such transitions and could be considered to be at the starting point of such a transition series. Such transitions are actually the normal condition among the members of each major class of rocks, i.e. igneous, sedimentary and metamorphic and, since all rocks can be understood as being derived from igneous rocks, the interrelationship of the members of this class will be explained in some detail.

IGNEOUS ROCKS

Igneous rocks are classified according to the chemical composition of the magma and the rate of crystallization, the latter being a function of the depth at which the magma has cooled and solidified. In Table 15, the chemical composition is given in the horizontal arrangement as 'acid', 'intermediate', 'basic' and 'ultra-basic' against the most characteristic mineral assemblages. The depth of solidification is given in the vertical columns: if the rock was derived from a magma which did not reach the surface and which, trapped at great depth, cooled slowly, coarse-grained

TABLE 15
Simplified classification of igneous rocks

TYPE		MINERAL ASSEMBLAGE	INTRUSIVE		EXTRUSIVE
			Plutonic	Hypabyssal	Volcanic
Acid		Quartz Orthoclase Mica or Amphibole	Granite	Quartz-Porphyry (Felsite)	Rhyolite
Intermediate		Orthoclase Amphibole	Syenite	Porphyry	Trachite
		Plagioclase Amphibole	Diorite		Andesite
Basic	Basic	Plagioclase Pyroxene	Norite	Diabase (Dolerite)	Basalt
	Ultra-basic	Pyroxene Olivine	Peridotite	Picrite	Olivine Basalt

plutonic rocks will have developed and if, trapped nearer to the surface, the magma cooled at a faster rate, then medium-grained hypabyssal rocks will have formed. Magma which has reached the surface and been extruded as lava which then cooled very quickly, makes fine-grained to glassy volcanic rocks develop.

The chemical composition of a magma can vary between types which contain much silica (SiO_2) and relatively few other components (acid), and those with relatively little silica and many other constituents (ultra-basic). All stages between the extremes of only silica, e.g. vein quartz, and no silica, e.g. certain types of ore, are possible and so are all transitions from one igneous rock to another. The same applies to the depth of solidification, where 'deep' and 'shallow' are only relative terms. Only typical representatives can therefore be given in Table 15 but each of these rocks could be surrounded by a great number of transitional types which would lead to the rocks given in the neighbouring fields.

The arrangement given in Table 15 is very crude and does not satisfy the modern petrological requirements for a classification of igneous rocks. The terms 'acid', 'intermediate' and 'basic' are used in this table solely with reference to the presence or absence of quartz but not with reference to the quantity of silicon dioxide, known as silica (SiO_2), in a magma from which the mineral quartz (SiO_2) develops under suitable conditions. In a cooling magma, silica first enters into chemical compounds with the other constituents, mostly metals, and only if the magma contains a surplus of silica, does this crystallize as quartz. Such a magma is then oversaturated with silica. A magma containing just sufficient silica to satisfy the maximum silica demand of the other constituents is saturated. A magma with insufficient silica is undersaturated and minerals typical for this condition, e.g. olivine, will be formed. From the above it follows that quartz can only develop in an oversaturated magma but there is no definite percentage of silica which defines the limits between the above grades of saturation. The reason for this is that the silica demands of the metal ions in a magma are not the same, e.g. potassium and sodium, in combination with aluminium, demand six times as much silica as do metals such as iron, magnesium or calcium alone, and the percentage of silica which makes a magma saturated at optimum, a condition which hardly ever occurs in nature, varies with the quantity of the various types of metal present. Modern classifications of igneous rocks are based on these grades of saturation with silica but such classifications are very involved, providing detail which is not required for the discussion of the properties of natural road construction materials; the crude classification given in Table 15 therefore appears sufficient.

In certain parts of Table 15, particularly amongst the basic and ultra-basic rocks, even more than one rock-name could be presented as typical. Norite has been used as the representative rock for basic plutonic rocks. This is correct for Southern Africa but would meet with objections in Europe and the USA where gabbro would be used instead. The difference between norite and gabbro lies in the crystallographic classes in which their pyroxenes develop. More basic plutonic rocks in Southern Africa are norite (orthopyroxene) and gabbro (clinopyroxene predominant) is less frequent while the opposite holds for Europe and the USA. In Southern Africa, therefore, norite is the type-rock and gabbro is a special type of norite, in contrast to Europe and the USA where norite is a special type of gab-

bro. To the engineer, however, these differences are of no significance because the engineering properties of both rocks are the same and the distinction between norite and gabbro has no practical impact. The two terms 'diabase' and 'dolerite' may also cause some confusion because there is little worldwide unanimity as regards the definition of these plagioclase-pyroxene rocks with ophitic texture. The general use in Southern Africa is to call all diabases of Karoo age 'dolerite', leaving 'diabase' to describe rocks of other geological ages with similar dyke and plate intrusions. Again, there is no difference between both types of basic igneous rock with respect to their suitability and performance as road construction materials.

SEDIMENTARY ROCKS

Similar transitions exist within the sedimentary rocks as can be seen in such very frequent series as sandstone – clayey sandstone – sandy shale – shale, or sandstone – calcareous sandstone – sandy limestone – limestone.

METAMORPHIC ROCKS

The situation is a bit different with the metamorphic rocks because they form part of genetic series where the individual type of metamorphic rock depends on the original rock and the conditions of temperature, pressure and infiltration of solutions which played a part in its formation. This transformation of rocks therefore takes place in an irreversible direction, e.g. the process during which shale is transformed to slate, sericite schist, phyllite, mica schist and eventually paragneiss could stop at any intermediate stage but could not be reversed. Such transitional stages exist, however, only along certain fixed directional lines of development and there are no transitions between such lines. Thus, shale changes into paragneiss, and granite into orthogneiss during metamorphism, and there are many intermediate types of rock between shale and paragneiss and between granite and orthogneiss, but there are none between paragneiss and orthogneiss. Intermediate stages do therefore exist between either igneous or sedimentary rocks and their metamorphic derivatives, but they do not exist between igneous and sedimentary rocks.

SCHEMATIC ARRANGEMENT OF ROCK TRANSFORMATION

There are many possible transitions within the separate groups of igneous and sedimentary rocks and well-defined, directed but irreversible transitions between any of these rocks and their metamorphic derivatives. All other transitions, e.g. between igneous and sedimentary, or sedimentary and igneous rocks, include an intermediate state, either weathering resulting in soil formation, or remelting (anatexis) resulting in a magma, where the character 'rock' is lost and a new beginning is made. These connections are illustrated in Figure 19.

The meaning of 'about 40 types of rock' which are used for road construction in Southern Africa may be clear now. Many more than these 40 types could be listed but they would refer to a vast number of intermediate stages and special rare
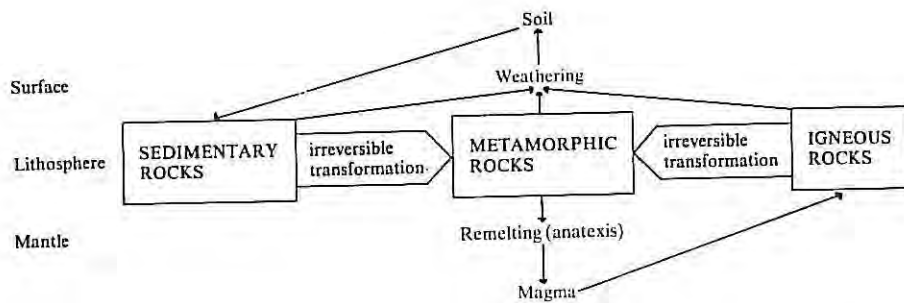
Figure 19: *Transitional connections between types of rock*

Because of the depth involved, only metamorphic rocks can be subjected to remelting (anatexis), in contrast to weathering which can affect all rocks.

types, many of which are only known to the expert (see Tables in Chapter 6). To name only 40 of the more commonly known types of rock therefore means a considerable simplification. These names have been selected as being representative of a number of sufficiently similar types of rock all of which have actually been used in road construction.

All of the 40 types of road-building rock differ sufficiently to be considered as separate entities. They differ in such details as mineral assemblage, structure, texture or mode of weathering, and each type should ideally be considered on its own merits and be used according to its own specifications. This is, of course, utterly impractical as it would require the assessment of the exact petrological nature of each natural road building material.

DURABILITY

When a natural road building material is selected, the property to be considered first is its durability. To decide about the durability of a material is to predict as accurately as possible what its condition will be at the time of expiry of the structural design life of the road. This decision depends on two criteria: the condition of the material at the time of construction and its intended use. A material can thus be durable if used for one purpose but not if used for another. The durability of a material is a natural property and follows from the tendency to change which is inherent in all rocks; this process has been described in detail under weathering. The rate of change varies, of course, and there are rocks which change fast and others in which the changes occur so slowly that none are visible during the life-span of a road. In addition, it must be realized that weathering (decomposition in a more pronounced way than disintegration) proceeds at an exponential rate. A natural

material at a less advanced initial stage of weathering, therefore, may change sufficiently slowly so as not to affect a road structure adversely, while another derived from the same type of rock but in a more advanced stage of weathering may, by the further progress of weathering, be rendered unsuitable for the intended purpose prior to expiry of the structural design life.

Weathering of a rock in its undisturbed setting proceeds at a certain rate. When the rock is removed from this setting, however, it is severely disturbed and once it has been laid in a road layer, it finds itself in a very different environment. The rate of weathering, decomposition in particular, increases. The amount of this increase depends on the local climate as expressed by the N-value, on the stage of weathering at the time the material was removed from its natural source, on the duration of dumping and on the position of the material within the road structure. In general, therefore, a natural material must not be considered as a 'permanent entity' whose properties change so little over the geologically short life of a road that they could not have an adverse effect on the road. To assess durability therefore means to assess how and at what rate those changes in a natural material that would occur as a matter of course can be expected to affect this material after its placement in a road layer.

Cases of very rapid decomposition have been observed in the past. In the late fifties a weathered dolerite whose condition was on the borderline between 'decomposed' and 'highly decomposed' (see Chapter 4 and Appendix 3) was used for the base of a surfaced road in the vicinity of Ladysmith in Natal. This road required excessive maintenance between six and twelve months after completion because the dolerite in the base had changed into a greenish, clayey material. Other less conspicuous cases involving decomposed dolerite have been reported by **Clauss** (1967).

The determination of durability is the prediction of a natural process as it is likely to continue after some disturbance. This means that durability cannot be 'tested' like many other properties of engineering materials. It is determined by making a projection, from a starting condition which can be described or even measured in some way, into the future on the basis of assumptions which one can only trust have been estimated correctly. Since there are no durability tests, durability must be predicted on the basis of such petrological properties that are sufficiently simple to be determined easily (see Chapter 7).

Durability is closely linked to the mode of weathering of rocks and therefore the difference between disintegration and decomposition is important.

The minerals in disintegrating rocks do not change which means that, as long as disintegration is the predominant mode of weathering, all types of rock are equally affected. The process and the results of disintegration are similar to those of crushing and in both cases the rocks' resistance to the natural or artificial breakdown, i.e. their crushing strength, varies with the type of rock and its position along the line of advancing disintegration (from the fresh rock to the final state of gravel- or sand-size material). This also means that all rocks can possess an equally low crushing strength but that their maximum possible strength varies with the rock type. This is obvious if one compares the original strengths of shale and quartzite, both undisturbed and fresh, with the strengths they decline to if they dis-

integrate: eventually neither of them possesses any crushing strength. The differences between the crushing strengths of fresh rock types is a function of the rocks' mineral composition, their structure and their texture, and the possible variations are such that some 'fresh' rocks are only suitable for use in selected subgrade and fill.

The effect of the component minerals on the strength of rocks is approximately proportional to their hardness. The presence of the hardest of the common minerals, quartz, is therefore of some significance: the more quartz a rock contains, particularly if it is well intergrown with itself or other minerals, the better such a rock generally resists disintegration or crushing. In crushing tests, therefore, pure silica rocks like quartzite or chert are amongst those with the highest crushing strength, but they are not the only ones at the top of the scale: certain fresh basic crystalline rocks in particular, e.g. dolerite, basalt or andesite, may occasionally be stronger than pure silica rocks.

The presence or absence of quartz is not the only criterion, and the structure of a rock, e.g. foliation, has a severely weakening effect on a rock even if quartz is the predominant mineral. Thus mica which occurs in layers between quartz, as it does in mica schist, makes the rock disintegrate or crush easily, breaking it down along the mica-rich planes.

The textural properties follow from the intergrowth of minerals, as has already been mentioned in connection with the composition of the minerals, their size and shape, and the cementing matrix in sedimentary rocks. Intergrowth of minerals as found in crystalline rocks adds to the strength of these rocks owing to the presence of particularly strong bonds between the individual minerals, in addition to the interlock. The size of the minerals affects disintegration and crushing strength inasmuch as large minerals part more easily from each other than small ones. The shape of the minerals, of course, again affects the interlock since angular minerals interlock far better than rounded ones. In all these aspects quartz is neither 'better' nor 'worse' than the other minerals.

The cementing matrix typical in sedimentary rocks may have either a beneficial or a weakening effect on their strength. The most common cementing matrices are argillaceous, calcareous, ferruginous or siliceous materials, arranged in approximate order of increasing strength and of the resistance which the rock offers to disintegration. Again silica (SiO_2) provides the highest resistance and strength, and sandstones with a siliceous cementing matrix, e.g. quartzitic sandstone, are the strongest and most resistant of sedimentary rocks.

The nett effect of the above properties of rocks is that a disintegrating rock develops first fissures, then cracks and eventually large fractures and that it breaks into pieces once cracking has penetrated deeply and frequently enough. Although a rock may appear to be quite solid and intact it will always, of course, contain fissures which peter out at some point and which are still so fine that they can only be detected under the microscope. Such fissures are the reason for the often unexpected multiple breaking of a rock after one blow with a hammer and they contribute to the weakness which is not easily detected.

Disintegration has a merely mechanical effect on a rock resulting in the decrease of its resistance to crushing and impact. Since it would be impractical to determine the degree of disintegration and to predict its further development by evaluating all contributing factors once the material has been laid in a road layer, the most suitable empirical method is to use crushing or impact tests; the 10% FACT is particularly recommended, especially if it is carried out on dry and wet material. Most mechanical damage to a road building material is done during construction by hauling, dumping and particularly rolling; the effect of traffic is less significant although by no means absent.

Things are different when decomposition plays a role. It has already been mentioned that decomposition requires special attention in Southern Africa (as it does in every country with a climate which stimulates chemical weathering) although it is restricted to rocks which contain minerals liable to decompose especially when they occur in areas where the N-value is less than 5. Decomposition depends almost entirely on the minerals contained by the rock, and structural and textural features are only involved in so far as they provide for, or prevent, the access of water to the rock.

Rocks which consist of primary minerals are particularly prone to decomposition and this gives quartz, the only important rock-forming primary mineral which does not weather chemically, its special position. It is obvious that the more quartz a rock contains, the smaller is its quantity of those minerals which will change into some type of clay. Rocks which contain quartz only, e.g. quartzite or vein quartz, do not decompose at all but only disintegrate. Every road engineer, and more particularly every materials engineer, should therefore become familiar with quartz at least to the extent that he can recognize this mineral in rocks and possibly also in a microscopic slide, although he need not necessarily be able to recognize and identify the other minerals.

In addition to the above, there are certain complex secondary minerals which may break down, i.e. decompose, into less complex ones if conditions are favourable. This applies in particular to clay minerals of the smectite group, e.g. montmorillonite, which may change into kaolinite especially in an environment where N is less than 2, and it also applies to kaolinite from which, amongst others, gibbsite ($Al(OH)_3$) may develop where N is less than 1.

THE RULE OF QUARTZ

For geochemical reasons, quartz occurs only in certain igneous rocks as shown in Table 15 where it is shown to be a characteristic rock-forming mineral of the acid types only in contrast to the other minerals most of which are characteristic of more than one of these types. Since quartz is a major constituent only in acid igneous and other related crystalline rocks, it is also an indicator of the type of clay mineral which is likely to develop from the decomposition of the other minerals of these rocks: if quartz can be recognized easily in a crystalline rock, the other minerals are in most cases of that type which decomposes to kaolinite; if quartz is scarce or absent, the other minerals will change into montmorillonite. Crystalline rocks which contain quartz as a major constituent are always light-coloured and the quartz-free crystalline rocks are mostly, but not always, dark. In general, therefore, quartz is a major component of the slowly decomposing acid crystalline

rocks whose other components change into kaolinite. Consequently, it gives these rocks a stable constituent in contrast to the quartz-free basic crystalline rocks which can change entirely into montmorillonitic clay soil.

In sedimentary rocks, quartz is the most important strength-giving mineral. These rocks do not decompose of course because they are made up of minerals which are themselves the result of past decomposition.

The only exception is diamictites, such as tillite or greywacke, which may be composed of a mixture of formerly decomposed and undecomposed minerals. The mode of weathering of these rocks depends, therefore, on their composition and the environment in which they occur and they may either decompose or, as is usually the case, disintegrate.

The following rule can therefore be formulated:

Quartz is the mineral which should be most generally known and recognized in a rock and possibly in a microscopic slide, too, because –

a) *Quartz does not decompose and it thus provides a 'stable skeleton' in decomposing rocks.*

b) Quartz is the hardest and one of the strongest of the common minerals.

c) Quartz almost never occurs in notable quantities together with mafic minerals and plagioclase, i.e. those minerals which change into montmorillonite during decomposition.

d) In rocks which contain much quartz the other minerals are either of the type which changes directly into kaolinite, or they are components of sedimentary rocks which are not subject to decomposition; the only exception is diamictites which always require geological advice.

This rule is the basis of the following classification of natural road building materials which reduces the number of about 40 such materials to nine groups or, if soils are included, to ten groups (see also Appendix 5).

THE SOUTHERN AFRICAN CLASSIFICATION
PRINCIPLES

In this classification, the Southern African natural road building materials are grouped according to their potential durability and their technical properties. This necessitates a break in several places with the traditional genetic classification as used in petrology (**Weinert**, 1968, 1969).

There are already a number of such classifications but most of them try to retain the genetic grouping although it is often of little relevance to engineering requirements: when used for engineering purposes, it is for instance of little interest that vein quartz belongs to the igneous, quartzite and hornfels to the metamorphic, and chert to the sedimentary rocks because what really matters is that these rocks are the most durable and, when fresh, amongst the strongest of all and that they are all equally suitable for various purposes in road construction. No such classification will ever be able to cover all possibilities fully and therefore this one is also not without a few exceptions in the detail.

The South African classification comprises nine groups of rocks which can be combined into three classes: the class of rocks which are liable to decompose, the class of those which only disintegrate and one class whose mode of weathering and whose road building properties depend entirely on their incidental composition. Soils may be regarded as the tenth group. The classification is based primarily on the presence or absence of quartz which has specific consequences in regard to the types of mineral associated with it in a rock, the mode of weathering and the potential durabilty when the material is used for road construction. If a roads engineer is able to recognize quartz, there is no need for him to identify the individual rock types as long as he can place his material in one of the nine groups.

CLASS: DECOMPOSING ROCKS

This class consists of the two groups of rocks which are liable to decompose in an environment where N is less than 5. These rocks are composed of minerals which have formed under conditions of high temperature, high pressure or both and which occur mostly as crystals, although a few rocks which contain much volcanic glass, i.e. amorphous matter, are included. In general these rocks have the most complex composition and may pose more problems regarding their durability than any others. As a class they provide the most widely used natural road building materials in Southern Africa.

Group: Basic crystalline rocks

In contrast to the usage in petrology, the basic rocks are described first and the acid rocks second because the basic rocks pose the greatest problems.

The road construction and performance properties of basic igneous rocks, especially those of Karoo dolerite, have already been described previously by the author (**Weinert**, 1964). It was stated then that what is said about dolerite applies equally to other basic rocks such as norite and basalt. The relationship between the N-value and the percentage of secondary minerals was also assessed in this report (**Weinert**, 1964) and this relationship with reference to road bases was expressed by what was at the time the 'soundness line' (Figure 11). 'Soundness' was later replaced by 'durability' (**Weinert**, 1974) and a curve for subbase materials was added (**Weinert**, 1968), based on general experience regarding the larger quantity of secondary minerals permissible in this structural layer.

The following individual members of this group have been used in road construction in Southern Africa:

TABLE 16
Basic crystalline rocks used as road building materials

Igneous			Metamorphic
Plutonic	Hypabyssal	Volcanic	
Diorite Gabbro Norite Peridotite Serpentinite	Anorthosite Diabase Dolerite	Andesite Basalt Phonolite	Amphibolite Greenschist

They have been used for the following purposes in Southern African road building:

TABLE 17
Use of basic crystalline rocks in Southern African road construction

Rock	Surfacing aggregate	Base	Subbase	Selected subgrade and fill	Gravel wearing course	Concrete aggregate
Amphibolite	+	+				+
Andesite	+	+	+	+	+	+
Anorthosite		+				+
Basalt	+	+	+	+	+	+
Diabase	+	+	+	+	+	
Diorite	+		+	+		
Dolerite	+	+	+	+	+	+
Gabbro	+	(+)	(+)	(+)	(+)	+
Greenschist	+	+	+	+	+	+
Norite	+	+	+	+	+	+
Peridotite		+	+	+	+	
Phonolite	+					+
Serpentinite	+	+	+	+	+	+

(+) probably used but recorded as norite.

It is common to these rocks that they contain no, or only very little, quartz and that montmorillonite is the principal clay mineral developed during their decomposition. Since all mineral components of these rocks are subject to decomposition, they may change so thoroughly into clay soil that none of the primary minerals is retained.

An exception is a special type of diorite, the quartz-diorite, which may contain sufficient quartz to make it one of its principal minerals. The other principal minerals in the rock are, however, plagioclase and amphibole both of which decompose to montmorillonite, i.e. the rock changes in much the same way as the other basic crystalline rocks.

Quartz-diorite has not yet been used in Southern African road construction; should this happen, however, it should be considered as a basic crystalline rock in spite of its quartz contents, and be treated accordingly.

The volcanic members of the group, andesite, basalt and phonolite, differ from the other members by the frequent presence of volcanic glass. Volcanic glass, being an uncrystallized, amorphous phase, is less stable than the crystallized phase of the same chemical composition. This instability makes such a glass weather,

either disintegrate or decompose, more easily than the corresponding crystallized minerals.

As with all other rocks, basic crystalline rocks disintegrate where N is more than 5. Hydromicas are the only secondary minerals which may develop in some quantity as long as N does not exceed 10. Where N is less than 5, the rocks decompose: montmorillonite is the predominant clay mineral and it often changes into kaolinite in the top portion of the decomposition profile if the surface gradient is more than 5° or if N is less than 2. Under certain hydrothermal conditions, however, montmorillonite may occasionally be formed already during the final cooling of the intruded magma (**Orr**, 1979). In dykes and sills this may be the case, in particular in the often coarser grained rock in the central and, where applicable, lower parts of deposits of more than 10 m thickness.

Basic crystalline rocks are the principal road building materials in the Republic of South Africa, the Transkei and in Lesotho. They are used in all other Southern African territories as well but on a much smaller scale. Consequently the Republic of South Africa, the Transkei and Lesotho are in a somewhat peculiar position: normally, acid crystalline rocks occur much more frequently on the surface of the Earth than basic ones. The South African peculiarity is caused by the very wide occurrence of deposits of the Karoo System, most of whose sedimentary rocks are less suitable for road construction than the intruded hypabyssal dolerite or the extruded volcanic basalt. These basic crystalline rocks of Karoo age have therefore always been used more often than the sedimentary rocks in spite of the durability problems associated with them, particularly where N is less than 5. Everywhere else on the subcontinent where Karoo rocks occur, dolerite and basalt are also used but not to the same extent as in the Republic of South Africa, Lesotho or the Transkei.

Other important basic crystalline rocks used for road construction are the andesitic lavas of the Ventersdorp System in the Western Transvaal and the North-Eastern Cape Province, andesite and diabase from the Pretoria Series of the Transvaal System in the Southern and Eastern Transvaal, the norite of the Bushveld Igneous Complex in large parts of the Transvaal and Bophuthatswana, and serpentinite of the Bulawayan and Sebakwian Formations between Bulawayo and Fort Victoria in Rhodesia. The other rocks of this group are only used on a very limited scale. Diorite and greenschist are used only in Rhodesia and phonolite only in South West Africa.

Use in road construction
Surfacing: Anorthosite and peridotite are the only basic crystalline rocks which have not been used for surfacing aggregate.

Surfacing aggregate must of course be obtained from fresh rock, i.e. rock which does not contain more than 15 per cent of secondary minerals. This rock must be crushed and the crushed stone should be clean and without an oxidized surface cover. The strength of the stone must be in accordance with specifications (**South African Bureau of Standards**, 1976) (see Table 33). It is recommended, however, that the dry strength of stone for surface treatment should be a bit greater than that for rolled-in chips while the wet strength must again not be less than 75 per

cent of the dry strength. The conversion from 10% FACT to ACV and vice versa is given in Chapter 7 and Appendix 6.

Most basic crystalline rocks are satisfactory in regard to their adhesion to bitumen and only occasionally volcanic members may possess rather smooth crushing surfaces, being thus more inclined to become detached. Occasionally, members of this group may be rather porous, especially those of volcanic origin and also rocks from the sill phase of dolerite as well as diabase dykes and plates. Although these pores are seldom interconnected, their numbers may be sufficient to affect the bitumen demand of the aggregate adversely.

Most basic crystalline rocks tend to polish because of the very similar hardness of their mineral constituents. Fully crystalline rocks, i.e. the plutonic and hypabyssal members which are rich in olivine (whose hardness is 6 to 7 on the Mohs scale), do not polish so much but olivine in basalt does little to improve the polished stone value (PSV) because the olivine crystals are clustered and inclined to break away from the rock easily once exposed to the atmosphere and traffic. A slight degree of decomposition raises the PSV of a stone because minerals which are already decomposing abrade faster than fresh ones. In this case, however, the presence of olivine would be of little if any advantage because, being the first mineral to decompose, it would be the principal contributor to the acceptable quantity of secondary minerals which must remain below 15 per cent.

The most suitable basic crystalline surfacing stone is therefore obtained from rocks which are fully crystallized, which contain between 10 and 15 per cent of secondary minerals, as determined microscopically, or, if the percentage of secondary minerals is less than 10, which contain more than 10 per cent of fresh olivine, i.e. ultra-basic rocks.

Base: All disintegrating basic crystalline rocks are sufficiently durable and can be used for natural gravel bases if they satisfy the design requirements in regard to grading and plasticity. The use of decomposing rocks, however, must be limited to those materials whose secondary mineral contents are in accordance with the climate at the site of use (see Figure 11).

Crushed stone bases are of course built of fresh rock, but the requirements for cleanliness do not apply as for surfacing aggregate. Thin oxidized layers on the surfaces of the stone provide no reason for rejection and the percentage of secondary minerals may be more than 15 as long as this percentage is in accordance with the local N-value (Figure 11) and the rock still lends itself to crushing. Otherwise, crushed basic crystalline rocks do not pose a problem with regard to durability.

The strength of the disintegrated or crushed rock must be more than 110 kN in the 10% FACT on the dry material and the wet test must give not less than 75 per cent of the strength obtained on the dry material. The corresponding Aggregate Crushing Value (ACV) of the dry material must not be more than 29 per cent (South African Bureau of Standards, 1976) (for conversion see Chapter 7 or Appendix 6).

Crushed or disintegrating basic crystalline rocks may be stabilized mechanically. The binder may be derived from the more weathered part of the materials deposit, it may be natural soil or it may be crusher sand. The presence of a natural binder whose composition is controlled by the specified plasticity index is often an ad-

vantage. The opinions of the different road authorities vary, however, as regards the use of soil or weathered material for the mechanical stabilization of such bases and generally those road authorities whose jurisdiction is mostly over dry areas (N more than 5), e.g. the Cape Province and South West Africa, are more open to the use of natural soil or weathered basic crystalline rocks for the mechanical stabilization of such bases than are those in the more humid areas.

Any base which has been stabilized mechanically is considered as untreated in contrast to a treated base where cement, lime or bitumen have been used as stabilizing agents. Cement is of course the most frequently used agent for the chemical stabilization of crushed stone bases. However, the addition of cement or lime to decomposing basic crystalline rocks whose percentage of secondary minerals is borderline in relation to the N-value (Figure 11) of the construction site is often of particular advantage because it retards further decomposition in the road structure (**Clauss**, 1967). The quantity required for this conservational effect is only about 0,3 per cent so that the use of any quantity for stabilization, to increase strength, is above that limit and thus of advantage. Since only 'weathered' (either disintegrated or decomposed) basic crystalline rocks may be used as a natural gravel for the base of any class of surfaced road, the initial consumption of lime (ICL) (**Clauss** and **Loudon**, 1971) will always be low and if it is above the critical 3,5 per cent, the material should no longer be classed as 'decomposed' ('weathered') but has reached the stage of 'highly decomposed'.

Subbase: Basically the same as for bases applies but the material may be more weathered. If disintegration predominates, i.e. if the N-value of the area is more than 5, and the material satisfies the specifications set for the job concerned, no problems will be experienced with its durability.

Where the rock is decomposed, i.e. where N is less than 5, the percentage of secondary minerals must be in accordance with the N-value of the construction site (Figure 11) but 10 per cent or more of secondary minerals than for the base is permissible. Again the addition of lime in small quantities helps to retard decomposition if a borderline material has to be used. Many highly decomposed basic crystalline rocks are initially lime-consuming (ICL) above the critical limit and, particularly if the construction material contains many fines more than 30 per cent of which pass the 2,00 mm sieve, and it therefore has to be stabilized with lime, the determination of the ICL is advisable. If strength has to be obtained rapidly, lime may be used to satisfy the lime demand and cement to achieve strength; both agents can be applied simultaneously.

Selected subgrade and fill: Basic crystalline rocks are durable in all stages of disintegration but in highly decomposed rock the montmorillonite content which is also indicated by high Atterberg limits must be controlled. Too plastic a material, if it has to be used, must be stabilized with lime to reduce plasticity and account must be taken of possible high ICL. There are no durability hazards.

Gravel wearing course: Basic crystalline rocks of subbase quality are the most suitable materials. Rocks at this stage of weathering often contain hard kernels which are a danger to moving traffic. Decomposed basic crystalline rocks whose percentage of secondary minerals is above 30 become slippery when wet especially in humid areas and near coasts with frequent mist. The acceptable quantity of

clay minerals in gravel wearing courses is very dependent on the local climate. This quantity is usually controlled adequately by the plasticity index, which is generally specified within the range of 8 to 18, values in the upper part of the range being applicable where N is more than 5 and in the lower part where N is less than 5. Where N is greater than 10, the plasticity index may even exceed 18. Compliance with these values automatically limits the acceptable quantity of minerals of the smectite group, especially montmorillonite, in decomposed basic crystalline rocks.

Concrete aggregate: Basic crystalline rocks make excellent coarse and fine aggregate in concrete whether the rock is fresh or slightly disintegrated. If an oxidized surface layer is present, the rock must be regarded with a bit more caution. It will do no harm in most cases, especially if the aggregate is fully embedded in the cement. Oxidation may be an indication, however, of the onset of decomposition and, especially if the environment favours decomposition and the basic crystalline rock concerned is of the volcanic type, e.g. basalt, undesirable components such as opal, certain zeolites and others may be released with undesirable effects on the concrete. Rock which is exposed to the surface of a concrete pavement will of course polish in the same way as stone in a bituminous surfacing. Serpentinite, particularly if used as fine aggregate, is likely to cause spalling of the concrete because of volume changes following changes in the moisture content. For more detail on the use of serpentinite as coarse or fine aggregate in concrete, see Chapter 10.

Group: Acid crystalline rocks

The following members of this group have been used for road construction in Southern Africa:

TABLE 18
Acid crystalline rocks used in Southern African road construction

Igneous			Metamorphic
Plutonic	Hypabyssal	Volcanic	
Granite Pegmatite (Aplite) Syenite	Felsite	Rhyolite	Gneiss

The gneiss in Table 18 is both orthogneiss and paragneiss and it is quite possible that a number of materials which have been recorded as granite are actually orthogneiss. This is indicated by question marks in Table 19 which lists the purposes for which the members of this group have been used. Inaccuracies of this nature are of course of little importance when the grouping under discussion is used for the classification and description of the materials.

The much smaller number of members in the group of acid crystalline rocks, if compared with the number in the basic crystalline rock group, follows from the large percentage of silica (SiO_2) which only allows for a considerably smaller number of other constituents in the rocks. The result is a restriction of the possible

TABLE 19
Use of acid crystalline rocks in Southern African road construction

Rocks	Surfacing aggregate	Base	Subbase	Subgrade and fill	Gravel wearing course	Concrete aggregate
Felsite	+	+	+	+	+	+
Gneiss	?	+	?	?	?	+
Granite		+	+	+	+	+
Pegmatite	+	+	+	+	+	
Rhyolite	+	+				+
Syenite	+	+				+

number of rock-forming minerals and thus of the number of possible rocks.

The common feature of all these rocks is the absence of a montmorillonite stage when they decompose. Orthoclase, often in the modification of microcline, is the predominant mineral in all these rocks and it changes directly into kaolinite once it has passed the stage of hydromica. Although it is the determining mineral of the group, quartz does not (or only as an accessory) occur in syenite which is composed of more than 80 per cent of orthoclase. Syenite, which is not a common rock and which has only been used for surfacing, base and concrete aggregate in the district of Worcester in the Cape Province, has been included in this group because of the way in which it decomposes, by which its potential durability and road performance are determined. The volcanic member of the group, rhyolite, may contain much volcanic glass in which may float rather large crystals of sanidine, a modification of orthoclase which occurs typically in volcanic rocks. This glass decomposes and turns into kaolinite more readily than do the crystallized minerals. Felsite consists of cryptocrystalline quartz and is a type of quartz-porphyry which lacks conspicuous phenocrysts of quartz. It is an extremely fine-grained rock whose grain can only be recognized under a microscope. Both rocks, rhyolite and especially felsite, may produce rather smooth crushing surfaces.

All acid crystalline rocks disintegrate where N is more than 5, and hydromica may develop in some quantity where N is between 5 and 10. Where N is less than 5, they decompose and kaolinite originates. Quartz, which comprises about 30 per cent by volume in most cases, does not decompose and, consequently, the residual soil is always a reasonably well-draining sandy kaolinite clay with the engineering properties of a silt rather than a clay. Internal drainage can lead to the loss of some of the clay components and this may result in a collapsible grain structure particularly where N is less than 2. For more details on this property see Chapter 9.

The durability of the acid crystalline rocks is best determined by crushing tests if the rocks are fresh or disintegrated, and by the determination of the percentage of secondary minerals if they decompose. The presence of the primary mineral quartz means that the percentage of secondary minerals will seldom exceed 70.

This reflects the true condition of these rocks when used for road construction and therefore, in contrast to an earlier opinion (Weinert, 1970), the percentage of secondary minerals in acid crystalline rocks should be determined and calculated in exactly the same way as is done for basic crystalline rocks.

Acid crystalline rocks, whose most important member is granite, are the most frequently used road aggregates in Rhodesia. Furthermore, they are about the only available natural road building materials in large parts of the Eastern Transvaal and are widely used in the Northern Transvaal, the North-Western Cape Province, Cape Peninsula, the Natal coastal belt and a couple of minor areas on granite outcrops, e.g. in Bophuthatswana. They are also widely used in South West Africa, especially in the central districts. It should be realized that many of these granites are certainly orthogneiss. Pegmatite, including the fine-grained variety aplite, has been used to a very limited extent, mainly for selected subgrade, in Rhodesia, felsite only in the Transvaal, rhyolite in Zululand (Natal) and, without success, in the Eastern Transvaal, and syenite, as already mentioned, only in the district of Worcester, Cape Province.

Use in road construction

Surfacing: Acid crystalline rocks make good surfacing stone if they are not too coarse-grained, porous or glassy. It may be noticed from Table 14 that pegmatite has not been used for this purpose. The properties of acid crystalline rocks as surfacing aggregate are very similar to those of basic crystalline rocks except as regards polishing. Only fresh rock, i.e. rock containing less than 15 per cent of secondary minerals, without oxidized layers on the surface must be used. Preference should be given to medium-grained rock which is more suitable for crushing than the coarse-grained varieties whose minerals may break loose individually. The strength of the stone must be in accordance with specifications (**South African Bureau of Standards**, 1976) (see Table 33). It is recommended, however, that the dry strength of stone for surface treatment should be a bit greater than that for rolled-in chips while the wet strength must again not be less than 75 per cent of the dry strength. The conversion from 10% FACT to ACV is given in Chapter 7 and in Appendix 6.

The adhesion of bitumen is satisfactory if medium-grained rocks are used but stripping is likely to occur on the faces of the large minerals in coarse-grained rocks or on the smooth surface texture of dense, glassy or fine-grained rock, such as felsite. The latter type of rock is scarce in Southern Africa. The porous varieties of rhyolite-type rocks occurring in Zululand, although they contain much volcanic glass, usually possess a surface texture which is sufficiently rough for satisfactory adhesion provided the rock is dry when laid; the performance of dense varieties of rhyolite is similar to that of felsite. Rhyolite may be porous, in which case it will absorb a relatively large amount of bitumen.

Most acid crystalline rocks do not polish excessively because of their quartz contents. The only exceptions are felsite and possibly syenite which, being composed of 80 per cent or more of orthoclase and no or only little quartz, possess polishing characteristics similar to those of basic crystalline rocks.

Base: Disintegrating acid crystalline rocks make good and durable natural gravel base materials which will not change during the life of a road. They must of course satisfy the design requirements as regards shape, grading, bearing capacity and plasticity. In decomposing rocks, however, in addition to the above requirements the percentage of secondary minerals must be in accordance with the local climate (Figure 11). The large quantity of quartz in most of these rocks means that the maximum quantity of secondary minerals hardly ever exceeds 70 per cent.

Acid crystalline rocks for crushed stone bases need not be as fresh as crushed rock for surfacing aggregate, and oxidized surface layers do no harm. The percentage of secondary minerals may be more than 15 but must be kept in accordance with the local N-value (Figure 11) as long as the rock can still be crushed.

The strength of the disintegrated natural gravel or the crushed rock must be more than 110 kN in the 10% FACT on the dry material and the test on wet samples must give more than 75 per cent of the dry test result. See Chapter 7 and Appendix 6 for the conversion from 10% FACT to ACV.

Disintegrated or crushed acid crystalline rocks always require some form of stabilization. The grading of crushed rock may need to be adjusted with the crusher sand obtained from crushing the fresh rock. The more weathered stages of the profile may be used with naturally disintegrated gravel. There are hardly any problems with regard to durability in areas where disintegration predominates, i.e. where N is more than 5. Where N is less than 5 and decomposition is the principal form of weathering, the durability must be ascertained, e.g. by the determination of the percentage of secondary minerals. Natural gravel may then be stabilized mechanically with some highly decomposed rock or residual soil may even be added as a natural binder. The kaolinite which is the natural soil binder produced by decomposing acid crystalline rocks will not cause problems as long as the bearing capacity of the material is kept under control, and the formation of more kaolinite from undecomposed orthoclase proceeds so slowly that the increase of the quantity of kaolinite will hardly be noticeable during the structural life of the road. It must be stressed again, however, that crushed rock should possibly only be stabilized mechanically with the crusher sand obtained from the crushing process.

The addition of lime, or more likely cement, when dealing with weathering acid crystalline rocks in treated bases retards decomposition. This allows the use of borderline material which, because of a slightly high percentage of secondary minerals in relation to the local climate, would otherwise be rejected. Initial consumption of lime (ICL) is unlikely to be critical in acid crystalline rocks and it need therefore not receive special attention.

Excessive mica, muscovite, as it occurs in many gneisses, causes the disruption of compacted layers if the muscovite is coarse, if it has become separated from the rock and if its quantity exceeds 10 per cent. If the muscovite is fine, or if there is less than 10 per cent of it, cement stabilization will counteract this effect. For more detail see Chapter 10.

Subbase: The material may be considerably more weathered than base material and in the case of predominating decomposition the quantity of secondary minerals must be controlled in accordance with the local climate (Figure 11). Otherwise everything said about natural gravel bases also applies to the subbase.

Selected subgrade and fill: Since kaolinite is the principal clay mineral forming

in decomposing acid crystalline rocks, these rocks are suitable as selected subgrade or fill in all stages of weathering, provided of course that they satisfy the design requirements as regards bearing capacity, grading and plasticity. Although they drain to some extent, provision for drainage must be made and damming up of water on the uphill side must be prevented.

Gravel wearing course: The general condition of the materials should be similar to that of subbase materials. The kaolinitic clay obtained from decomposing acid crystalline rocks is a good binder for the stronger, hard material, mostly quartz and undecomposed feldspar, provided its quantity does not exceed 10 to 20 per cent – nearer to the higher value where N is more than 2 and to the lower one where N is less than 2 – because too much clay gives rise to the risk of potholing and makes the riding surface slippery when wet. Hard weathering kernels must be avoided because of the hazard to moving traffic. Excessive loose muscovite will be detrimental to compaction and may also cause slipperiness. The most suitable quantity of the clay component of the material and thus the acceptable degree of decomposition is controlled by the plasticity index which may vary according to the local climate. This plasticity index is usually specified as between 8 and 18, the lower part of this range applying to areas where N is less than 5 and the higher part to areas where N is more than 5, while a plasticity index of 18 may even be exceeded where N is more than 10.

Concrete aggregate may be more weathered than aggregate for bituminous surfacings. All acid crystalline rocks make good concrete aggregate and the same as for crushed stone bases applies since there is no problem with polishing. With the exception of syenite, all members of the group may however contain undesirable quantities of reactive silica (**Oberholster** *et al*, 1978) which may cause serious cracking of the concrete. For more details refer to Chapter 10.

CLASS: DISINTEGRATING ROCKS

The next few groups are composed of rocks which only disintegrate. This means that decomposition is a process which does not need to be considered for the assessment of the road building quality of these rocks. This does not exclude the possibility that certain rocks in this class may contain primary minerals which became part of these rocks before the minerals could decompose. Examples of such rocks are arkose and arkosic sandstone, both sandstones which contain a varying quantity of orthoclase, i.e. that type of feldspar which resists decomposition more than any other. Other sandstones may contain considerable quantities of muscovite which has a similarly high resistance to decomposition. Both primary minerals decompose very slowly and the N-value applies to them in the same way as it does to acid crystalline rocks. The decomposition of these minerals, however, leads to kaolinite and this as a clay mineral adds a binder to the disintegrated rock and seldom causes problems.

Any primary minerals in rocks of this class never occur in such a quantity that they would greatly influence the durability and road performance of those rocks. Certain sedimentary rocks that may well contain large quantities of primary

minerals because of the particular way they were formed, and which may consequently decompose almost as intensely as the crystalline rocks, are not included in this class.

Most members of this class are sedimentary rocks but a few igneous and metamorphic rocks are also included. Two of the groups within this class comprise the majority of the clastic sedimentary rocks. 'Clastic' is a term which refers to the mode of formation and is derived from the Greek verb meaning 'to break up'. These rocks consist of detritus which has been transported, mostly by water but also by wind, ice or even volcanic action, and then deposited elsewhere. The mechanical means of transport is the decisive criterion by which a clastic rock is distinguished from other types of sedimentary rock. Such other sedimentary rocks are formed by chemical precipitation, e.g. many limestones, or by the accumulation of organic residues, such as coal or oil. However, if one of these other types of sedimentary rock, e.g. a limestone, disintegrates and the resulting debris is transported and deposited elsewhere, this new rock will also have become a clastic sedimentary rock.

The most widely distributed clastic sedimentary rocks are sandstone and shale and they are also the most important members of the groups of arenaceous and argillaceous rocks respectively, whose road building properties are described below. Basically, sandstone is composed of quartz grains and shale of clay minerals but the mode of formation of these rocks allows for all possible transitions between these two. In addition, as far as sandstones are concerned, there is a wide range of possible grain sizes which makes the rocks vary from the very coarse-grained conglomerate, through gritstone and ordinary sandstone to the very fine-grained siltstone whose quartz grains may be within the grading range of 'clay size'.

The denomination of the transitional stages and especially of the very fine-grained types has often been uncertain or caused confusion. In the terminology used in Southern Africa as well as in many other parts of the world, a rock name often incorrectly used is 'mudstone' (**Underwood**, 1967). Rightly, a mudstone should be an unstratified, massive rock composed predominantly of clay minerals. The very small grain size of many siltstones, however, often makes it difficult to distinguish between mudstone and siltstone without employing sophisticated methods such as the inspection of microscopic slides. This sort of more detailed investigation has often shown that what has been assumed to be a mudstone is actually a siltstone (**Price** *et al*, 1969; **Olivier**, 1976).

So far, the need to distinguish between Southern African sandstones, siltstones, shales and mudstones as well as the numerous possible transitional stages has almost only stemmed from their impact on civil engineering projects. Recently, **Holleman** (1975), **Purnell** and **Netterberg** (1975) and **Weinert** (1975) have discussed certain aspects of such a distinction. Overseas, work on these problems has taken place over many years and relevant references would have to go back over decades (**Underwood**, 1967). Two more recent reports, both concerned with the properties of shale as a road construction material, which have been presented by **Dea** (1972) and **Reidenouer** *et al* (1974) contain comprehensive bibliographies which not only refer to publications on shale or mudstone but also include references to related rocks such as siltstone.

A crude, rather primitive method can be used in the field to identify rocks which contain appreciable quantities of clay (i.e. more than approximately 30 per cent like shale or mudstone) in order to assess their suitability as road building materials: if a piece of shale or mudstone in the above sense is put on the wet tongue, it produces an effect similar to that obtained with blotting paper. This effect, which is due to the fast absorption of moisture by the clay minerals, is not felt with other types of rock and it also disappears gradually with the increase of induration in a clay-containing rock. Experience of the proper 'feel on the tongue' may best be gained by trying the method with a piece of rock which is known for certain to be a shale and comparing this with the feel of a clay-free rock. Disintegration is the overriding form of weathering of the members of this class. This does not make them and their road performance very dependent on the local climate.

Group: High-silica rocks

The following members of this group have been used in road construction in Southern Africa:

TABLE 20
High-silica rocks used in Southern African road construction

Igneous	Sedimentary	Metamorphic
Vein quartz	Chert	Hornfels Quartzite

They have been used for the following purposes:

TABLE 21
Use of high-silica rocks in Southern African road construction

Rock	Surfacing aggregate	Base	Subbase	Selected subgrade and fill	Gravel wearing course	Concrete aggregate
Chert		+	+	+		
Hornfels	+	+				+
Quartzite	+	+	+	+	+	
Vein quartz	+	+	+	+	+	+

These rocks consist mostly of quartz or opal which may amount to nearly 100 per cent in chert, quartzite and vein quartz. A somewhat different condition, which will be discussed below, makes hornfels also a member of this group. In any environment all these rocks will only disintegrate and during this process they eventually break down to gravel-, sand- or silt-size material. When fresh they are extremely strong, in fact the values obtained on fresh high-silica rocks in the 10% FACT are among the highest and in the aggregate crushing test among the low-est. Their absolute durability does not necessitate much testing and the rocks are often too strong to be worked economically, particularly if wanted for use in layers below the surfacing.

Crushing tests obviously provide the most adequate means of testing the suitability of high-silica rocks for all possible uses in road construction. These tests will also indicate that once high-silica rocks have started disintegrating the rate of break-down tends to increase rapidly. This rate of disintegration makes it advisable to limit the decrease of the wet strength in the 10% FACT to the usual 75 per cent. This is, however, of little relevance to the chemical, or rather mineralogical, durability of the rocks and it may appear to contradict statements made about indurated argillaceous rocks. The possibility of such rapid disintegration in quartz grains or larger masses of quartz or opal, has been studied by **Smalley** (1966), **Moss** (1973) and others with the purpose of understanding the presence of the large quantities of silt in particular but also sand and their lithified derivatives on Earth. Approaching disintegration can often be recognized by one or more of the following properties:

a) The rock looks matt milky-white, this colour being caused by numerous fine, criss-crossing cracks.

b) The rock breaks into numerous larger and smaller chips at only one blow with a hammer.

c) The surface of the rock feels sandy or rough even if no grains can be rubbed off with a finger.

Hornfels, in petrology, refers to any rock which has been formed by thermal metamorphism. This is a transformation which rocks experience if they are intruded and 'baked' by a magma. Thermal metamorphism does not change the chemical composition of the heated rock but does change its mineral composition. The resulting rock therefore depends on the composition of the affected original rock and the temperature involved; a very large variety of new rocks can therefore develop. The term hornfels in connection with the Southern African road construction materials is very much restricted in its meaning and refers only to argillaceous rocks, such as shale, slate or mudstone, which have been altered by thermal metamorphism. If affected by heat, these rocks are silicified, the silica being obtained from the clay minerals and occasionally, to a lesser degree, from silica emanating from the intruded magma. The other chemical constituents of the argillaceous rocks recrystallize into minerals which typically result from this type of metamorphism, e.g. andalusite or cordierite, and which are fairly stable under surface conditions. The result of the transformation is a very fine-grained, dense and strong rock which, although its silica content may be as low as 60 per cent by mass, is as durable a natural road building material as the other members of this group.

High-silica rocks have been widely used in the Republic of South Africa and Bophuthatswana. Much less use has been made of them in Rhodesian and South West African road construction, solely because of a lack of local sources. Only quartzite and to a lesser degree vein quartz have been used in Rhodesia, and quartzite at many isolated places and chert only in the north of South West Africa.

In the Republic of South Africa, the principal source of chert is the Transvaal Dolomite and that of hornfels the thermal metamorphic Malmesbury shale. Other

sources of hornfels, unfortunately also often called 'shale', occur in the Dwyka, Ecca and Beaufort Series in the vicinity of dolerite intrusions. There are several sources of quartzite but most of the material is obtained from the Witwatersrand System and the Pretoria Series, and vein quartz is confined to acid intrusions particularly in the Northern Transvaal and Rhodesia. The use of quartzite in the northwestern and central areas of the Orange Free State is based on the mining operations in the Odendaalsrus-Welkom area whence the material is obtained for surfacing and concrete aggregate, and base. A considerable portion of the 'quartzite' used in the southern parts of the Cape Province is actually quartzitic sandstone and it will be discussed with the group of arenaceous rocks. Similarly, as will be pointed out under 'surfacing' below, 'Witwatersrand quartzite' is a mixture of a number of very different rocks.

Use in road construction

Surfacing aggregate: Crushed high-silica rocks are always sufficiently strong for use as a surfacing aggregate and the specifications of the **South African Bureau of Standard** (1976) apply (Table 33). The percentage difference between the dry and wet test results is of so little relevance that it may even be neglected for the assessment of the long-term durability. This difference serves, however, a useful purpose when a choice between high-silica rocks from different sources has to be made. The material with the smallest difference between dry and wet test results (preferably with a wet test result of more than 75 per cent of the dry test result) should be selected to keep degradation during construction under control. Chert has not yet been used for surfacing.

When crushed, high-silica rocks are likely to produce a smooth surface texture and stripping of bitumen is very likely to occur. It may be argued that stripping does not necessarily occur when Witwatersrand quartzite is used as a surfacing aggregate. 'Witwatersrand quartzite' is however, not the name of a particular type of rock but that of a succession of rocks of the Witwatersrand System within which true quartzite plays an important role but which also contains quartzitic sandstones, hornfels and highly indurated shales. As a result, the adhesion of the aggregate which is generally called 'Witwatersrand quartzite' to bitumen varies considerably.

The resistance of high-silica rocks to abrasion is more or less uniform over the whole surface of the crushed stone and they therefore polish very slowly. Although the stone thus attains an overall polished surface under traffic, its great resistance makes the binder abrade so much faster that the road surface remains sufficiently rough even if wet not to become a danger to traffic. With respect to Witwatersrand quartzite, the comments made in the preceeding paragraph apply to Witwatersrand quartzite in the sense that some of these rocks, e.g. the quartzitic sandstones, are very resistant to abrasion and polishing while others abrade and polish more easily or more evenly.

Lower layers of the road structure: Since high-silica rocks are durable in all stages of disintegration, there is no need for separate discussions of their performance when used for *base, subbase, selected subgrade* or *fill.* As long as these rocks satisfy the grading requirements they can be used. High-silica rocks often require an adjustment of their grading and often need to be stabilized with cement or bitumen. Cement-stabilized crushed rock has often been used for the bases of freeways or other heavily trafficked roads. This has been done with varying success but failures were not due to the weakness of the aggregate but to unbalanced structural design.

The crushing strength of high-silica rocks when used crushed or natural for base should be in accordance with the specifications of the **South African Bureau of Standards** (1976) although the specified values are of little relevance in assessing the durability of these rocks (Table 33).

Road layers where high-silica rocks are used as a natural gravel, blended with soil, drain easily if the soil binder is not too fine, i.e. not too clayey or silty.

Special precautions are required when certain quartzites of the Witwatersrand System are used in any layer of a road structure. Some of these rocks contain sulphide minerals: the best known is iron pyrite (FeS_2), but there are also marcasite (FeS_2, a modification of pyrite) and chalcopyrite ($CuFeS_2$) which is also known as copper pyrite. All these sulphides decompose very easily and fast if exposed to the atmosphere and during the decomposition process first sulphuric acid, and then a variety of sulphate salts develop. Sulphuric acid is of course deleterious to cement and certain sulphate salts, which migrate upwards through the road layers, cause the disruption of the bond between the base and the surfacing. For more details see Chapter 10.

Gravel wearing course: High-silica rocks have only been used on a limited scale in gravel wearing courses. The quality of the material should be equivalent to that of selected subgrade rather than that of subbase. The high-silica rocks can only be used for gravel wearing courses if they have disintegrated to sand size and if some soil binder is available. Coarse material is very rough on tyres, if it is sufficently angular to be rolled and compacted successfully, while rounded grains, which are not so rough on tyres are difficult to roll.

Concrete aggregate: Generally, high-silica rocks make satisfactory concrete aggregate. Sulphide minerals do not pose a great problem, particularly if the aggregate has either not been dumped for more than a few days prior to use, or if it has been dumped long enough, so that all or the majority of the sulphide minerals have decomposed. A short period of dumping would do no harm because most of the stone is eventually embedded in the cement and thus cut off from the effect of air and water. The occurrence of brown, rusty spots on the surface of a concrete pavement, which develop when these minerals decompose, is of little importance because aesthetic problems that are of concern in the building industry do not apply to carriageways, although such spots may well be undesirable in bridges. It should be realized, however, that many high-silica rocks, chert and hornfels in particular, may contain varying amounts of opal, chalcedony or other types of reactive silica (**Oberholster** *et al*, 1978) which may cause expansion if the rocks are used as an aggregate especially with a high-alkali cement.

Group: Arenaceous rocks

The following members of this group have been used in Southern African road construction:

TABLE 22
Arenaceous rocks used in Southern African road construction

Sedimentary	Metamorphic
Arkose Conglomerate Gritstone Sandstone	Mica schist

These rocks have been used for the following purposes in road construction:

TABLE 23
Use of arenaceous rocks in Southern African road construction

Rock	Surfacing aggregate	Base	Subbase	Selected subgrade and fill	Gravel wearing course	Concrete aggregate
Arkose				·		+
Conglomerate		+	+	+		
Gritstone		+	+	+	+	
Mica schist			+	+	+	
Sandstone	+	+	+	+	+	+

The arenaceous rocks have in common that they consist principally of quartz; this is in the form of sand grains in the sedimentary rocks and is obtained by recrystallization of silica released from metamorphically altered clay minerals in the case of mica schist. Most siltstones are also members of this group (**Price** *et al,* 1969) since very fine quartz is their principal component. If the contents of other constituents, mostly clay minerals, exceeds 35 per cent then siltstones, as well as any other member of the group of arenaceous rocks, become members of the group of argillaceous rocks.

The sedimentary rocks of this group disintegrate when weathering which means that they eventually return to what they were originally: sandstone and arkose become sand, conglomerate and gritstone become gravel. Mica schist, however, which is a metamorphosed shale, becomes micaceous sand. All arenaceous rocks may contain a limited number of primary minerals (**Holleman,** 1975), which are liable to decompose, e.g. orthoclase in arkose and pebbles of any type of rock in conglomerates or micas, especially muscovite in mica schist. Their limited quantity and the fact that they mostly turn into kaolinite, however, can only have a beneficial effect on the weathered rock in which a useful natural binder then develops. Disintegration is therefore virtually the only form of weathering of these rocks and their suitability for road building is determined by the degree to which their strength has decreased.

The sand grains in the sedimentary rocks are held together by a cementing matrix which may be argillaceous (clayey), ferruginous (mostly iron hydroxide), calcareous (calcium carbonate or occasionally dolomite) or siliceous (amorphous silica). These cementing materials are arranged above more or less in increasing order of the overall strength of the rock: for example, quartzitic sandstone, the member with the siliceous matrix, is the strongest member of the group of arenaceous rocks (**Yedlowsky** and **Dean,** 1961).

The bond in a mica schist is different from that in the sedimentary members of the group. This rock is composed of muscovite, or occasionally biotite, and quartz, and the mica often occurs in undulating, wavy layers with intercalated lenses of quartz. The bond between these minerals is similar to that of crystalline rocks and strong intergrowth is prominent between the quartz crystals, while the bond between quartz and muscovite is weak. As a result these rocks are inclined to break into platey pieces along the planes of muscovite if the rock disintegrates or is crushed, and eventually a very mica-rich sand develops. The micas may of course decompose where N is less than 5, the muscovite changing into kaolinite, and the biotite, through illite, into montmorillonite. The rate of decomposition of both types of mica is rather slow, especially that of muscovite which is by far the more common mica mineral in mica schists. It can be assumed that the condition of a disintegrated mica schist, determined by whatever test may be considered appropriate, will not change noticeably during the structural design life of a road.

Since disintegration is the prominent form of weathering in all environments, strength tests are the most suitable means of assessing the durability of arenaceous rocks which is solely mechanical durability anyway. The 10% FACT on dry and wet samples is particularly recommended.

Sandstone is the arenaceous rock most widely used in road building in the Republic of South Africa, the Transkei and Lesotho, and arkose, gritstone and conglomerate have only had limited local application. Sandstone has only been used occasionally in South West Africa and Rhodesia. In Rhodesia a silty sandstone has been used for the lower layers of roads in the Melsetter area. Arkose has been used occasionally in Natal and the Transvaal and on a slightly larger scale in the northern parts of Rhodesia, particularly in the Alaska and Magula areas. Mica schist has only been used in the central parts of South West Africa.

Use in road construction
Surfacing aggregate: The only arenaceous rocks which have been used successfully as a surfacing aggregate are quartzitic sandstones, i.e. those whose cementing matrix is siliceous. Many of the surfacing stones which are commercially called 'quartzite' are in fact quartzitic sandstones, particularly those which have not raised problems regarding adhesion to bitumen. A quartzitic sandstone which satisfies the strength requirements as specified by the **South African Bureau of Standards** (1976) (Table 33), i.e. which does not disintegrate too much, makes a better surfacing aggregate than true quartzite because the difference between the

strength of the cementing matrix and that of the sand grains prevents the development of smooth crushing faces. Satisfactory adhesion between the stone and the binder is the result. The hardness of the siliceous cement, which is mostly opal, is about 6 on the Mohs scale and that of the sand grains is 7; consequently, quartzitic sandstones polish less than quartzite.

Base: Arenaceous rocks, especially sandstone, have been used more frequently for the base than for any other layer of a road. Besides its grading, the suitability of an arenaceous rock for road bases depends on its strength. This strength is determined primarily by the cementing matrix and it can therefore vary considerably through the whole range of members this group. There is no difference between the stability of crushed or naturally disintegrated arenaceous rocks. A minimum strength of the dry material of 110 kN, which is approximately equivalent to an Aggregate Crushing Value (ACV) of 29 per cent, has been specified by the **South African Bureau of Standards** (1976), provided that the strength of the wet material is not less than 75 per cent of that of the dry one. This strength limit may appear to be low for arenaceous rocks thus involving the risk of selecting materials which will disintegrate easily into a sandy material during compaction. Earlier recommendations (**Weinert**, 1969) set the limit at 180 kN to produce 10 per cent fines from the dry material. This value was considered too high, however, since a number of suitable arenaceous rocks could in this way have been excluded from use as a base material. Following from the above, it is suggested that when intended for use in a road base disintegrated or crushed arenaceous rocks require a force of at least 140 kN (ACV of 27%) to produce ten per cent fines, provided the wet test result is not less than 75 per cent of the dry test result. It is supposed that this limit will exclude arenaceous rocks with a clayey, ferruginous or calcareous cementing matrix unless they have been subject to some degree of induration. Only if the cementing matrix of the rock is silica should a strength of 110 kN be accepted as indicating the suitability of an arenaceous rock for the base.

Whether stabilization, mostly with cement, is required, depends on the cementing matrix and the type of road in which the material is to be used. Arenaceous rocks with a cementing matrix other than silica often produce a natural binder during weathering which is sufficient to bind the rock pieces. Initial consumption of lime (ICL) does not play a role when these rocks are used because the clayey matrix which might absorb some lime, particuary if the clay mineral is montmorillonite, does not occur in such quantity as to affect the amount of lime or cement added noticeably. Cement-stabilized disintegrated arenaceous rocks with a low grading modulus (**Kleyn**, 1955), i.e. those with a relatively high percentage of fines, may develop severe shrinkage cracks but they may also develop strengths which considerably exceed those obtained after seven days' curing in the laboratory.

Certain arenaceous rocks (conglomerate more than sandstone) contain sulphide minerals, particularly pyrite (FeS_2). As long as these minerals are fresh and as long as they do not start decomposing, they will do no harm. If they have a chance to decompose, however, as would be the case after rocks containing sulphide minerals have been dumped for a few years allowing the access of water and air, they provide a considerable hazard to the overall strength of a base and to the bond between base and surfacing. This is due to the development of sulphuric acid with deleterious effects on stabilizing agents like lime or cement, and to soluble sulphate salts which are the final decomposition products. In particular the arenaceous rocks of the Transvaal System and older should be checked for their possible sulphide mineral contents and if these minerals make up more than 1 per cent of the volume of the rock and more so if they are distributed evenly throughout it, the rock should be rejected. For more detail refer to Chapter 10.

Excessive muscovite, particularly if it is present as coarse mica (diameter more than 0,5 mm) and amounts to more than 10 per cent of the total volume of the base material, can disrupt compacted layers (**Tubey**, 1961; **Tubey** and **Bulman**, 1964). More detail is given in Chapter 10. Disintegrated mica schist has, therefore, never been used for bases in Southern Africa.

Subbase, selected subgrade and fill: All types of arenaceous rock have been used in the lower layers of a pavement although less frequently than in the base. All disintegrated and highly disintegrated arenaceous rocks of satisfactory grading, plasticity and bearing capacity are sufficiently durable for this purpose. If the rock lacks natural binders, as would particularly be the case with rocks with a siliceous cementing matrix, cement stabilization is required for its use in the subbase and blending with soil for its use in selected subgrade and fill. Arenaceous rocks with a clay matrix, especially if the latter is composed largely of montmorillonite, may have to be stabilized with lime to reduce their plasticity. Some such lime-stabilized sandstone subbases have stabilized so well that they developed shrinkage cracks which passed right through 100 mm of bitumen-treated base. The reason for this cracking was probably an excess of chemically active silica which made the lime have an effect akin to that of cement.

Most of the arenaceous rocks drain freely and drainage will only be impeded if the rock is silty or clayey, especially if the clay is montmorillonite.

Arenaceous rocks have not been used much for *gravel wearing courses*. Those used have always been highly disintegrated rocks whose properties are very similar to those of sand into which they quickly turn during grading, rolling and under traffic. Grain size distribution and plasticity, the latter especially in areas where N is more than 5, are the principal selection criteria. Those varieties of arenaceous rock which provide some natural binder, i.e. which possess a cementing matrix other than silica, are more suitable than the others.

Arenaceous rocks have only been used as *concrete aggregate* in the South Western, and the Central and Eastern Cape Province and very limited use has been made of arkose in Rhodesia and in the district of Oudtshoorn. All precautions required in concrete technology with regard to sulphides and sulphates, clay minerals and reactive silica are also needed when arenaceous rocks are used for concrete pavements and concrete structures of roads (**Fulton**, 1977; **Oberholster** *et al*, 1978).

Group: Argillaceous rocks

The following members of this group have been used for road construction in Southern Africa:

TABLE 24
Argillaceous rocks used in Southern African road construction

Increasing regional metamorphism ←	Sedimentary ←→	Increasing thermal metamorphism →
Phyllite←Sericite schist←Slate	Shale Mudstone	'Baked' shale

The position of a siltstone depends on the mineralogical nature of its constituents: if these constituents are mainly very fine quartz, the rock must be considered an arenaceous rock. The inclusion of a siltstone in the group of argillaceous rocks is then based on the same criteria in regard to the quantity of clay and related minerals as set out below (**Price** *et al*, 1969).

The use of these rocks in road construction is restricted to the layers below the surfacing and to gravel wearing courses. It is not possible to tabulate the uses of the individual members of this group because the transition from one type to another is so gradual that what is recorded as one type in one area, may be recorded as another type in a neighbouring one. This problem of deciding on the correct denomination for rocks as vaguely defined as the argillaceous ones can be overcome by using a group name as is done here. It should also be remembered that hornfels, which must not be included in the group of argillaceous rocks, is occasionally called 'shale', e.g. the Malmesbury 'shales' or shales of the Karoo System in the vicinity of dolerite intrusions, and that in this way the impression may be gained that certain 'shales' have even been used as surfacing or concrete aggregate.

The gradual transition between the members of this group is a result of their principal constituents, the clay minerals. The rocks are derived from the lithification of clay and most of them contain some quartz. Just as there is a transition between the individual members of the group, there is also a transition between shale or mudstone, and sandstone. In petrology, the boundary between shale and sandstone is usually set at 50 per cent clay and 50 per cent sand by volume, and adjectives like clayey or sandy are used for conditions in the vicinity of this boundary. For engineering purposes, however, a rock which contains 50 per cent by volume of clay must be treated as a shale because the sand grains are no longer in a position to touch each other and the rock therefore has no strong and stable skeleton. If it were supposed that the sand grains were all ball-shaped, of equal size and equally distributed in the rock, the maximum quantity of clay still allowing for mutual touch of the sand grains would be about 45 per cent. At this quantitative limit, however, the rock has little strength because the clay component still acts as an effective lubricant. It is safe, therefore, to consider each rock which contains more than 30 to 35 per cent by volume of clay as an argillaceous rock for engineering purposes (**Pettijohn**, 1957).

The complexity of clay minerals is the reason for the development of different types of rock if argillaceous rocks are metamorphosed. While an arenaceous rock becomes quartzite and a carbonate rock is marble during most types of metamorphism, the changes which occur in argillaceous rocks differ according to the conditions of regional and thermal metamorphism. Dynamic metamorphism, which is associated with zones of folding or faulting, produces rocks which go under the general name of mylonites. Again these are either quartzite or marble in the case of arenaceous or carbonate rocks, although they are texturally different, while the rocks produced from shale resemble those obtained by regional metamorphism but again with textural distinctions. Thermal or dynamic metamorphisms are local occurrences in contrast to regional metamorphism which affects rocks over large areas.

During regional metamorphism, *shale* and *mudstone* are compressed and dehydrated to a noticeable extent and both change into *slate*. The principal components of slate are still clay minerals some of which have started, however, to change into sericite, a mixed mineral of the hydromica type. Sericite is mostly concentrated along the bedding planes of the rock which thus obtains a typical 'silky' lustre. Further increase of pressure causes continued dehydration and growth of sericite which eventually becomes so prominent that the rock is called *sericite schist*. Besides sericite, this rock still contains clay minerals. Further metamorphism leads to *phyllite*, a rock which still contains a few clay minerals but in which the sericite changes into muscovite and the thin lenses of quartz, which develop through the recrystallization of liberated silica, start giving the rock a wavy structure.

Continued metamorphism leads to rocks which are devoid of clay minerals, *mica schists* and *paragneiss*, and which therefore do not belong to the group of argillaceous rocks.

Thermal metamorphism, caused by the heat generated by an intruding magma, only affects the rocks in the immediate vicinity of such an intrusion. Argillaceous rocks adjacent to magma change into hornfels which is a new rock, free of clay minerals and therefore not a member of the group. The transitional stage between hornfels and an unaffected argillaceous rock is commonly known as *'baked' (indurated) shale* which, because it still retains clay, is an argillaceous rock. Such slightly metamorphic rocks are only of limited occurrence, their relative importance in Southern Africa, particularly in the Republic of South Africa, being due to the frequent occurrence of such intrusions especially in the Transvaal, the Nama and the Karoo Systems.

Shale and mudstone gain strength during metamorphism. In the process of regional metamorphism, however, this increase of strength, which is less than that occurring during thermal metamorphism, is counteracted by extreme foliation which causes only very flaky stone to be created during disintegration or when such rock is crushed. In sericite schist and phyllite, the increasing amount of muscovite has a further adverse influence on the engineering properties of the rocks.

Thermal metamorphism, on the other hand, has a favourable influence on argillaceous rocks because with increasing degrees of induration, the foliation disappears until the transformation into an entirely massive hornfels is complete. Apart from shale and, rarely, mudstone which have not been metamorphosed, 'baked' (indurated) shale has therefore been used in road construction more often than other types of argillaceous rock.

The principal clay minerals which determine the performance of argillaceous rocks in road construction are the hardly expansive kaolinite and the very expansive montmorillonite. Both are the products of the decomposition of crystalline rocks but since kaolinite is a more stable end-product of decomposition than montmorillonite, it is more frequently the principal clay component of argillaceous rocks than montmorillonite. In general the Southern African montmorillonitic shales and mudstones are of Karoo age and younger, while kaolinite is found as a predominant clay mineral in argillaceous rocks of all ages. The fact that a very large part of the Republic of South Africa is covered by Karoo sediments explains the frequent surface outcrops of montmorillonitic shales; with increasing geological age illite and chlorite may take the place of the above clay minerals (**Purnell** and **Netterberg**, 1975). The mudstones, particularly those of the Beaufort Series of the Karoo System, are notorious for their fast disintegration into small cube-like pieces after exposure to the atmosphere. The dimension of these pieces is mostly 5 to 10 mm. The mechanism of this disintegration is not yet fully understood and a number of different explanations have been suggested. Most likely each of the proposed mechanisms, such as the presence of expansive clay, overconsolidation, or capillarity affected by variations in relative humidity, play a certain role in the disintegration of exposed mudstone but, depending on the composition and the structural properties of each occurrence, one of them may be more prominent than the others. It should be realized that many mudstones do contain a large amount of clay minerals and these are rightly considered to be members of the group of argillaceous rocks. There are many other occurrences, however, which have been called mudstone although they should rather have been classified as siltstones and been considered to belong to the group of arenaceous rocks (**Price** *et al*, 1969; **Olivier**, 1976).

Argillaceous rocks only disintegrate, turning into some type of clay and a varying amount of sand or silt (quartz) and possibly muscovite, depending on the composition of the orginal rock and the degree of metamorphism attained. The durability of these rocks when used for road construction therefore depends mostly on their strength, and crushing tests are suitable to determine this. Since argillaceous rocks can be very weak, the 10% FACT is the obvious method to use. However, the peculiarities of this group necessitate a somewhat more involved set of quality standards than those required for any other group. The greatest gain of strength is experienced by those argillaceous rocks which have been subject to thermal metamorphism. Once such a rock has attained the crushing strength of 250 kN in the 10% FACT, it is turning into hornfels, a member of the group of high-silica rocks (see page 162) which does not require specification for durability.

The peculiarities of the origin, the composition and the metamorphic alterations of argillaceous rocks demand (when they are to be used for road construction) an approach to the strength requirements which differs somewhat from that for other rocks. **Loubser** (1967) has already observed that the strength of shale, which had been used successfully in road bases, is such that almost always more than 200 kN is required to produce 10% per cent fines from the dry material: successfully used subbase material requires at least more than 150 kN on the dry material. At a strength of about 250 kN, argillaceous rocks are changing into hornfels

and, as a consequence, the ranges of strength within which these materials are suitable for use in a base or subbase is comparatively small. The latter fact has been observed previously by **Hatcher** (1963) in New South Wales although crushing tests have not been used there for quality assessment.

Following the work of **Loubser**, subsequent experience has shown that, unless they are intended for selected subgrade or fill, the critical strength limit for argillaceous rocks should be set at 160 kN on the dry material in the 10% FACT, provided that the wet test gives at least 75 per cent (120 kN) of the dry test result. As the dry test results increase, however, the permissible difference between dry and wet test results increases at such a rate that at 220 kN in the dry test, that is when the rock is already highly indurated, the acceptable wet test result is only 60 per cent of the dry test, i.e. a wet test result of 132 kN: in other words the acceptable wet test values vary only by 12 kN (Figure 20).
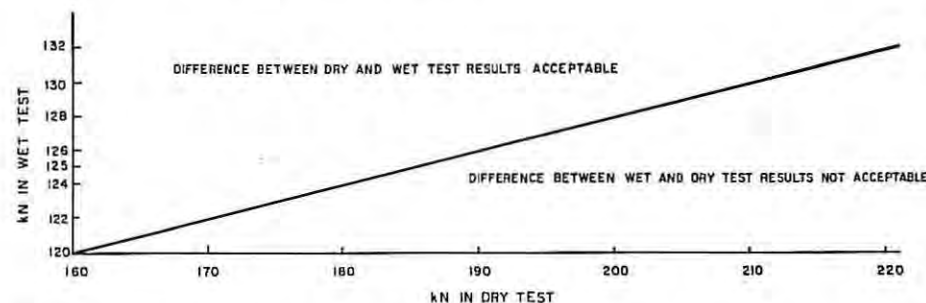
Figure 20: *Acceptable wet test results for argillaceous rocks in relation to dry test results*

The order of variation of the actual forces on the wet material in comparison with the permissible percentage drop is shown in Table 25.

TABLE 25
Acceptable percentage of wet test results in relation to dry test results on argillaceous rocks

Dry test result	Acceptable percentage of wet test result	Wet test result
160 kN	75,0	120 kN
170 kN	71,8	122 kN
180 kN	68,9	124 kN
190 kN	66,3	126 kN
200 kN	64,0	128 kN
210 kN	61,9	130 kN
220 kN	60,0	132 kN

Over a total range of 12 kN in the wet test there is thus only a deviation of ± 6 kN from the arithmetic mean of 126 kN. Since this deviation lies within the experimental error it is sufficiently safe to specify the following 10% FACT results on

argillaceous rocks if they are intended for use in the base or subbase of a road:

Dry test result: more than 160 kN

Wet test result: more than 125 kN

This makes the requirements in the lower strength range in the dry test a bit stricter, thus adding a little safety to the weaker rock, and it allows a bit more play for a stronger material approaching the condition of a hornfels. It may be suggested that only a wet test result of 125 kN should be specified for argillaceous rocks. This must not be done, however, since it may result in the acceptance of very slightly indurated rock of this group whose strength is for example 140 kN in the dry, and 125 kN in the wet, test. Such a material would probably not stand up to the handling during the construction of a road.

The increasing deviation of the wet test limits from increasing dry test values is a peculiarity of the argillaceous rocks which stems from the way in which these rocks change under the influence of metamorphism. Regionally metamorphic argillaceous rocks, i.e. the series from shale or mudstone through slate and sericite schist to phyllite, will hardly ever attain a strength of more than 200 kN in the dry test and they are, therefore, hardly ever suitable for anything more than subbase and are mostly used only for selected subgrade and fill. Thermal metamorphic argillaceous rock, i.e. 'baked' shale which has not yet turned into hornfels, is that type to which the scale from 160 kN to 220 kN in the dry test relative to a fixed wet test result of 125 kN particularly applies.

Thermal metamorphism should have a similar effect on an arenaceous rock with a clay matrix, i.e. a clayey sandstone, and it actually does. It should be remembered, however, that a rock whose clay content is greater than 30 to 35 per cent should be regarded as an argillaceous rock for engineering purposes and that the clay content in arenaceous rocks, as defined in this context, is relatively low. Under the conditions of thermal metamorphism, therefore, all arenaceous rocks turn into quartzite with a varying amount of impurities. The durability and strength of the intermediate stages are governed by the interaction of the bulk of the sand grains, i.e. quartz minerals, in an often greatly different matrix.

Argillaceous rocks have been used most frequently for road construction in the Orange Free State, the Cape Province, the Transkei and Natal. Most of these materials have been derived from the Karoo System. These rocks have been used much less frequently in the Transvaal where the source is mostly the Pretoria Series of the Transvaal System. Very little use has been made of these materials in Rhodesian and South West African road construction.

Use in road construction

Argillaceous rocks have only been used for base, subbase, selected subgrade, fill and for gravel wearing courses. The so-called 'shale' which has been used as surfacing or concrete aggregate, such as Malmesbury 'shale' or Karoo shales from the vicinity of dolerite intrusions, is actually hornfels.

Base: Karoo shales and mudstones have been used locally for base in Natal, particularly in the south of the province. In the Central and Eastern Transvaal, use has been made of shales of the Pretoria Series and argillaceous rocks of various ages, again mainly shale, have been used for base at isolated sites in South West Africa. In all these cases the rock has been indurated by diabase or dolerite intrusions. In Rhodesia, cement-stabilized phyllite has been used by consultants in the base of lightly trafficked roads around Salisbury.

Only fairly highly indurated ('baked') argillaceous rocks are sufficiently strong and durable for use in the bases of roads which carry traffic that is not too heavy. The minimum strength should be 180 kN in the 10% FACT on the dry material provided that the crushing strength of the wet material is not less than 125 kN; such a material is durable in any environment and sufficiently strong for use in the base of a road. Mutual rubbing of the rock pieces, particularly during construction operations but also under traffic, makes fine rock-powder develop and cover the aggregate. This powder, especially if it gets wet, acts as a lubricant which makes the pieces of aggregate move under traffic, thus reducing the overall bearing capacity of the base without affecting the strength of the individual rock pieces. Chemical stabilization of such bases is therefore advisable. Certain indurated argillaceous rocks may absorb lime, particularly those in which montmorillonite is the predominant clay mineral or in which some amorphous silica is present, and the determination of the initial consumption of lime (ICL) is advisable.

Subbase: All types of argillaceous rock have been widely used for subbase in the whole southern part of the Cape Province, more precisely south of 30 °S, from the Atlantic coast to the border of Natal. Extensive use of these rocks has also been made in Natal. Their use in the Transvaal has been confined to the districts of Marico, Pretoria, Bronkhorstspruit, Belfast, Lydenburg, Potchefstroom and Heidelberg. Phyllite has been used to a limited extent in the north of Rhodesia and the north of South West Africa.

All argillaceous rocks used for subbase should be indurated, although a lesser degree than is required for base is acceptable. Strength and durability are again best determined by the 10% FACT. The minimum force required to produce ten per cent fines should be 160 kN or more in the dry test and in the wet test not less than 125 kN. Fine material which could act as a lubricant may form but this would happen mostly during construction and would not be affected much by traffic. The decision to stabilize or not must therefore depend on the overall design to keep the bearing capacity and strength of the subbase in balance with those of the base. If stabilization with lime or cement is considered, account must again be taken of possible initial consumption of lime (ICL).

Selected subgrade and fill: All sorts of argillaceous rocks have been used for this purpose in nearly the whole Cape Province south of 30 °S, and in large parts of Natal, but only on a limited scale in the Southern, Central and Eastern Transvaal. Except for the north-western parts, argillaceous rocks have been used everywhere in the Orange Free State for fill but not for selected subgrade; considerable use has also been made of mudstone in the east of the province. In South West Africa shale has been used in the south and in the north and, in addition, sericite schist and phyllite have been used in the Otavi-Grootfontein area. In Rhodesia, very limited use has been made of shale in the Melsetter area.

No strength limits need be specified. It is important, however, to provide efficient drainage which may pose some problems if the material contains much montmorillonite. Special precautions are required for high embankments to prevent sli-

ding and reference should be made to TRH 10 (**National Institute for Road Research**, 1975) for design. Fills must be so constructed that no water can dam on the up-hill side.

Gravel wearing course: In the Cape Province and Natal the use of argillaceous rocks, almost only shale, for gravel wearing courses coincides with the areas of use of such rocks for selected subgrade. In the Transvaal they have been used at various localities in the western, central, eastern and southern parts of the province. Argillaceous rocks have also been used for wearing courses of gravel roads in the south of South West Africa and, extremely seldom, on private rather than public roads in Rhodesia. Argillaceous rocks have not been used for gravel wearing courses in the Orange Free State.

The condition of argillaceous rocks for gravel wearing courses should be the same as for selected subgrade material rather than as for subbase material. Most argillaceous rocks of subbase quality are too hard and they would be rough on tyres and also cause a hazard to moving traffic in other ways. These rocks do not make ideal gravel wearing courses anyway. All of them, even phyllite, get slippery when wet, particularly if the rock is montmorillonitic, and they are very dusty when dry because of the large quantity of fines which develops under traffic. Corrugation in the form of transverse ripples is not as pronounced as when less cohesive material is used and some types of argillaceous rock develop a crust, often called a 'blad' by road workers, which allows for relatively smooth driving. If this crust breaks, however, potholing and rutting occur quickly and severely. Loss of shape frequently occurs if the road is wetted thoroughly. Since relatively soft rock must be used, gravel wearing courses made of any type of argillaceous rock require continuous maintenance even if the techniques that are available to combat corrugation, rutting and dustiness are employed.

Group: Carbonate rocks

Carbonate rocks were previously called 'calcareous rocks'. 'Carbonate' appears to be a better choice, however, since materials like magnesite ($MgCO_3$) may be included in this group at a later stage.

The following members of this group have been used in Southern African road construction:

TABLE 26
Carbonate rocks used in Southern African road construction

Sedimentary	Metamorphic
Dolomite Limestone	Marble

All these rocks are composed chiefly of carbonate minerals, i.e. minerals which contain the carbonate (CO_3) complex. The principal minerals are the calcium carbonate $CaCO_3$, calcite, and the calcium-magnesium carbonate $CaMg(CO_3)_2$,

These rocks have been used for all purposes of road construction:

TABLE 27
Use of carbonate rocks in Southern African road construction

Rock			Subbase	Selected subgrade and fill	Gravel wearing course	Concrete aggregate
Dolomite	+	+	+	+	+	+
Limestone		+	+		+	+
Marble	+	+	+	+	+	+

dolomite. The pure magnesium carbonate $MgCO_3$, magnesite, may form rock masses occasionally and this material has also been used in road construction locally but it will be discussed with the Group of Metalliferous Rocks in which category it must stay until such time as more is known of its performance, when it may be transferred to the Group of Carbonate Rocks.

Most carbonate rocks are chemical or organic precipitates which have been deposited under water; some are composed largely of shells or shell fragments. During metamorphism other than thermal these rocks change into marble. This change only affects the texture which becomes coarse-grained and crystalline in contrast to the very fine-grained, cryptocrystalline sedimentary rocks. There is also a rare igneous carbonate rock, carbonatite, which is not included in this discussion because it has not yet been used for road construction in Southern Africa; its engineering properties would, however, make it a member of this group.

The conditions of origin of the sedimentary members of the group, limestone and dolomite, in clean, still, deep water results in remarkably pure rocks which consist of more than 80 per cent of carbonate minerals with only incidental grains of sand or clay.

When weathering, carbonate rocks only disintegrate. In contact with water they may dissolve, however, but this should not be considered to be decomposition in the sense of the change from primary to secondary minerals in crystalline rocks. If the carbonate in solution is precipitated again, it becomes what it was before, namely a carbonate, and no truly new minerals are formed although dolomite is reprecipitated as calcite. The peculiarity of disintegrating carbonate rocks is their lack of a true weathering profile which can be subdivided into several stages. Disintegration of these rocks proceeds in basically the same way as that of all other rocks. During the break-up of the rock, however, water penetrates along the cracks and dissolves the rock as it does so. The dissolved material is carried away. This is not very noticeable as long as the individual rock blocks are still large but the effect of solution becomes increasingly marked with the decreasing size and consequently increasing surface area of the disintegrating rock. Depending on the climate, an equilibrium is attained between the average rainfall and the minimum

size of rock pieces which make up the bulk of the top portion of the disintegrated rock. This equilibrium is such that the minimum size decreases with increasing rainfall because of the rate of solution and wash-out of finer material.

The consequence of this mode of weathering of carbonate rocks is that comparatively solid rock lies at or near the surface. The rock is only covered by a thin layer of topsoil which is composed of rubble derived from the underlying rock. The size of the rock pieces depends on the prevailing environmental conditions, and these pieces are mixed with blown-in sand and insoluble residues, e.g. wad in pockets in dolomite, derived from the carbonate rock itself. Opening borrow pits in such rocks is difficult. Where N is less than 5, the best sites are small sinkholes which are not too deep and do not endanger working gangs, or the valleys of streams and small rivers. Where N is more than 5, carbonate rocks are often covered by a thick sheet of calcrete which makes the actual carbonate rock rather inaccessible for the purpose of road construction. The lack of extensive layers of gravel or sand-size material on top of carbonate rocks is the reason why they have mostly been used as crushed rock for surfacing stone and concrete aggregate.

Carbonate rocks have been used mostly in the north-east of the Cape Province, the south-west of the Transvaal, in Bophuthatswana, the Witwatersrand area and the Eastern Transvaal, i.e. in the zone of outcropping Transvaal dolomite. In the Central Transvaal, local use has been made of limestone and, in the district of Groblersdal and locally in South West Africa, of marble which is mostly metamorphosed dolomite. Limestone and dolomite of the Nama System have also been used locally in the Western and South-Western Cape Province. In Rhodesia, limited use has been made of dolomite of the Lomagundi Formation in the central-northern parts and of limestone of the Umkondo Formation as a concrete aggregate in the Melsetter and Chipinga areas of the east. In South West Africa dolomite of the Otavi series and marble of the Khomas series, both series of the Damara System, have been used successfully for crushed stone bases in the northern and central parts respectively and some have performed satisfactorily without any binder.

Use in road construction

The almost complete lack of economic quantities of disintegrated rocks is the reason why carbonate rocks have mostly been crushed for use as surfacing or concrete aggregate. This lack of limestone or dolomite rubble may be a bit surprising especially if one considers the vast quantities of talus gravel in the younger mountain ranges of the Earth, e.g. in the Northern and Southern Alps. The reason for this lack probably lies in the interaction between climate and topography in Southern Africa: carbonate rock areas with sufficient relief also receive so much moisture especially in the form of rain during the warm season that solution is easy and fast.

Surfacing aggregate: Every carbonate rock which is suitable for crushing is also suitable for surfacing aggregate. The strength of carbonate surfacing aggregate should be in accordance with the specification issued by the **South African Bureau of Standards** (1976) (Table 33) which will automatically restrict the use of many

such rocks to roads carrying traffic which is not too heavy. The rocks are, however, peculiar in regard to other performance characteristics. The fact that carbonate rocks are the best surfacing stones as far as *adhesion* to bitumen is concerned has often been quoted as one of the strongest arguments in favour of the chemical concept of adhesion of stone to bitumen (see Chapter 6) because, due to the nearly complete absence of silica (SiO_2), they were regarded as equivalent to extremely basic rocks. The other component which has been assumed to promote weak adhesion, the carbon dioxide, is never more than 50 per cent by mass if shown in a chemical rock analysis as CO_2. Carbonate rocks never produce smooth crushing faces and perhaps this, together with the fact that they are slightly less positively charged than most other rocks, is the reason for their satisfactory to strong adhesion with any type of bituminous binder.

In regard to *polishing* carbonate rocks are among the least suitable of all surfacing stones. The great uniformity of these rocks, being composed almost exclusively of either calcite with a hardness of 3 (Mohs scale) or dolomite with a hardness of 3,5 (Mohs scale), makes them abrade easily and evenly and the small amount of sand grains which occasionally occur in these rocks does not improve the condition. Moreover, the bond between the carbonates and sand is not particularly strong and the sand grains are easily torn off under traffic, always leaving only the carbonate portion of the stone exposed to abrasion.

The degree of possible polishing naturally depends on a number of independent variables such as traffic, weather and the amount of grit and dirt on the road or clinging to the tyres. Occasionally the texture of carbonate rocks, especially limestone and dolomite, can be another modifying factor: if such rocks are uniformly cryptocrystalline, as they very often are, they will polish more severely than those which contain the calcareous shells of microfossils or pieces of the broken calcareous shells of mussels or other such larger creatures. **Underwood** *et al* (1971), in their study of the polishing of aggregates of Texas (many of which are carbonate rocks) found some relationship between the insoluble residues, i.e. constituents left behind after the solution of all carbonate by diluted hydrochloric acid, and the polished stone value (PSV) of the rock. The PSV increased slightly when the quantity of sand-size insoluble residue exceeded 5 per cent. Moreover, they found that if an aggregate of carbonate rock was blended with another aggregate less inclined to polish, the PSV increased linearly, e.g. a mixture of 50 per cent limestone with a PSV of 30 and 50 per cent quartzitic sandstone with a PSV of 60 gives an overall PSV of the mixture of 45.

Lower layers: Carbonate rocks have been used on a limited scale for the lower layers of road structures only in the Republic of South Africa.

Base: Where they could be obtained in sufficient quantity, disintegrated or crushed carbonate rocks have been used on a limited scale for base. The durability is always satisfactory and the specifications of the **South African Bureau of Standards** (1976) should apply for the strength (Table 33). However, because of the complete lack of natural binders, either blending with soil or, if reasonably graded, stabilization with cement is normally required although some crushed marble in the central parts of South West Africa has performed satisfactorily without the addition of any binder.

Subbase: Carbonate rocks have occasionally been used for subbase, particularly where N is more than 5, when they have reached a stage of disintegration which could be classified as highly disintegrated. Durability and strength are satisfactory. The grading mostly needs to be adjusted by blending with soil. Occasionally, the material has been stabilized with cement.

Selected subgrade and fill: On a limited scale, highly disintegrated carbonate rocks have been used for selected subgrade and fill, particularly where N is more than 5. Blending with soil is always necessary but the material does not pose problems with regard to drainage.

Gravel wearing course: Locally, mostly where N is more than 5, highly disintegrated rocks have been used for gravel wearing courses. The material consists of stone mixed with soil and it does not make a wearing course which is pleasant to ride on.

Concrete aggregate: Crushed carbonate rocks have been used successfully for concrete aggregate almost as widely as they have been used for surfacing stone. The overall strength of the concrete, either in a concrete pavement or as structural concrete, is not very great because of the rather low strength of these rocks which is seldom more than 200 kN on the dry material in the 10% FACT but many such rocks satisfy the requirement of 110 kN for concrete surfaces subject to abrasion (**South African Bureau of Standards**, 1976). These rocks do not contain undesirable contaminants.

CLASS: SPECIAL GROUPS OF ROCKS

Two groups of road building materials, the diamictites and the metalliferous rocks, do not fall naturally into either of the preceeding two classes because their general characteristics differ to some degree. They have, therefore, been kept separate and each will be discussed on its own merits because they have virtually nothing in common.

Group: Diamictites

The members of this group, only one of which has so far certainly been used for road construction in Southern Africa, are greywacke (**Buckland**, 1967), tillite (**Weinert**, 1967) volcanic breccia and volcanic tuff (**Roper**, 1973). Of these, tillite, especially that of the Dwyka series has definitely been used as natural road building material while greywacke, which occurs in several stratigraphical units, may have been used occasionally but may have been recorded as sandstone: the description of greywacke as a 'sandstone' when used for road construction will often be quite acceptable because it is difficult to give an exact definition of these rocks which occupy a position somewhere between sandstone, conglomerate and arkose. They may consist of a great variety of rock and mineral fragments which makes them rather variable engineering materials and, as shown by **Holleman** (1975) in his investigation of Beaufort sandstones, they may also occur more widely than is normally thought.

Greywacke develops under conditions of fast erosion and is deposited as a sedi-

ment mostly in deep troughs, e.g. in geosynclines, from high land on the margins of the troughs. Tillite is the lithified morainic material, till, deposited by glaciers. Volcanic breccia and tuff are pyroclastic sediments which form from the rubble and ash blown out of a volcanic vent immediately prior to, or during, a volcanic eruption. These breccias or tuffs may occur as the filling of such vents or may be deposited around the centre or centres of volcanoes.

Tectonic breccias and their most extreme form, mylonite, may also be considered as members of the group of diamictites. In general, however, the composition of such breccia remains that of the original, although crushed, rock to which silica or perhaps felspathic or calcitic components may be added on reconstitution. When a basic crystalline or argillaceous rock is crushed tectonically to such an extent that it becomes a mylonite, certain new minerals such as chlorite or sericite may form. Nevertheless their road building properties are still the best assessed according to the requirements of the original group.

The name 'diamictite' (**Flint**, 1971) refers to those rocks which were previously called 'cataclysmic rocks' (**Weinert**, 1974). This group comprises a rather large variety of rocks. They are all characterized by very variable and incidental composition resulting from their particular mode of formation. The term 'diamictite' (or 'diamicton') is fairly new in petrology and it should be used in the sense defined by **Flint** (1971), while 'mixtite' (**Schermerhorn**, 1966; **Kröner**, 1977) should be used to refer to those members of the group which were formed by relatively long-lasting processes of sedimentation, e.g. tillite or greywacke.

Although originally formed by considerably different mechanisms, these rocks are commonly composed of everything which happened to be in the path of the creating force, (water, ice or gas). This force acted in such a way that, besides fractionation, no process of weathering had time to alter the individual components of the rock. The result is unsorted rocks composed of minerals and rock fragments which are either liable only to disintegrate (corresponding to the rocks of the Class of Disintegrating Rocks) or which will decompose in a suitable environment (corresponding to the Class of Decomposing Rocks). These differences in the mode of weathering can exist within a single stratigraphic rock layer as well as in successive superimposed strata and, since they have a considerable influence on the performance of the rocks in a road, they must be given the relevant attention.

The great and often sudden variation in the composition and, consequently, the properties of the rock necessitates a separate assessment of the quality of each new deposit which is intended for use in road construction, even if it is the stratigraphical continuation of another deposit whose properties are already well known. Similarly, when one is working downwards in a quarry or borrow pit (see *Plate 24*), the quality of the material must be controlled continuously. Superimposed deposits of different ages, even if they all belong to the same type of rock, e.g. tillite, may differ to such an extent that, what applies to one layer may not be relevant to the one lying immediately above or below it. In nearly all cases expert knowledge is required to assess these variations and continuous geological advice is more necessary when dealing with materials of this group than when dealing with any other group.

Tillite is the only member of the group which has definitely been used as a road

building material in Southern Africa. Greywacke and the volcanic materials have been used in other parts of the world and the problems encountered with them there have been very similar to those experienced with tillite in Southern Africa.

Tillite has been used for all purposes of road construction although mostly for those which require a high-quality material. So far, the use of tillite has been restricted to the south of South West Africa, to the southern parts and a few isolated spots in the west of the Cape Province, and also in the vicinity of the Natal coastal anticline. Most of these rocks are Dwyka tillite and belong to deposits which were laid down by floating ice. Since they were deposited under water which prevented them from being washed out by subglacial or periglacial streams and rivers, they are essentially fine-grained, massive rocks composed mainly of sand and clay (illite and montmorillonite being relatively prominent) mixed with fragments of all sorts of other minerals and rocks of all sizes. Such minerals and rocks are called erratics. These erratics particularly quartz, quartzite, quartzitic sandstone and acid crystalline rocks, may be so plentiful in places that for practical purposes the road building properties of the tillite become those of high-silica, arenaceous or acid crystalline rocks. In most cases, however, the road building properties of tillites lie between those of the arenaceous and indurated argillaceous rocks.

Tillite which was deposited on dry land may also have been used for road construction, particularly in the south of the Transvaal. This material was often washed out severely by the subglacial and particularly the periglacial streams and rivers prior to lithification. This resulted in the loss of most fines, mostly from the fine sand fraction downwards and consequently the deposits decreased considerably in thickness. The material which was left behind and eventually lithified was a rather coarse-grained sand; when this has been used for road construction it has probably been regarded as, and correctly treated as, a sandstone. Only close inspection will reveal that this material contains an appreciable quantity of other minerals and rock fragments, particularly high-silica and acid crystalline rocks, noticeably less basic crystalline and arenaceous rocks, and hardly any material from the other groups.

Use in road construction

Particularly in Natal, tillite has been classified commercially as 'blue', 'first brown' and 'second brown'; these classes correspond to 'fresh', 'weathered' and 'highly weathered' respectively. The difference between blue and first brown tillite is only the degree of oxide staining on the faces of cracks. Very few such cracks are present in the massive blue type while the first brown type is cracked and the faces of the cracks of the rock, which has now disintegrated into blocks, are often severely stained with iron hydroxide. The interior of the blocks and pieces of first brown tillite is as fresh as the blue tillite and crushing tests on both dry materials can be up to 270 kN in the 10% FACT. In the second brown stage, the rock has weathered completely, changing mostly into a clayey soil, the detailed composition of which depends on the composition of the fresh rock.

The presence of clay minerals in an otherwise hard and strong rock calls for a slight revision of the permissible decrease of the crushing strength of the wet material. If such a tillite is wetted, some moisture is absorbed by the exposed surfaces of the clay minerals, thus weakening a thin surface layer rather than the bulk of the stone. The total number of fines produced will then be slightly greater than it would be if no such clay minerals were present. The specification of a permissible decrease of 25 per cent of the force required to produce 10 per cent fines from the wet material, as has been found useful for most other rocks, will therefore rule out a number of useful tillites. It has been found that the permissible decrease of force required to produce these 10 per cent fines from wet tillite can be reduced to 30 per cent, i.e. the wet test on tillite must give at least 70 per cent of the dry test result.

Surfacing aggregate: Crushed fresh tillite makes a good surfacing stone if it is not too water-absorbent. Its crushing strength should be 220 kN for rolled-in chips, 220 kN for surface treatment and 170 kN for any type of bituminous mixture. The wet 10% FACT on tillite should never give less than 70 per cent of the dry test results. The above values are slightly higher than those specified for most other groups by the **South African Bureau of Standards** (1976) but they are recommended to allow for the possibly weakening effect of clay minerals which are prominent constituents in many tillites (Table 33).

Certain tillites may absorb water because of their clay contents and because of fine voids. This may lead to stripping of the stone if a method other than laying a hot bituminous mixture is applied. Otherwise *bitumen adheres* well to tillite because the surface texture of the stone is always rough.

Tillite is among the least liable to *polish* of all Southern African road building materials. The polished stone value (PSV) of these rocks often varies between 70 and 80, i.e. almost reaching the limit of PSV of more than 80 at which the stone is considered not to polish at all.

Tillite, being composed of many big erratics of other rocks, produces crushed stones which consist largely of broken pieces of these rocks. Such crushed stone must be treated, and its durability must be assessed, according to the group of which the particular erratic is a member.

Base: Crushed fresh and weathered, i.e. first brown, tillite is suitable for crushed stone bases and weathered tillite can be used for natural gravel bases. Both materials lack natural binders and stabilization with cement is advisable. Up to 10 per cent of highly weathered (second brown) tillite can be allowed as a natural binder for the first brown (weathered) tillite when used in the natural gravel base of a road carrying medium to low traffic. The quantity and uniform intermixture of the highly weathered rocks must, however, be controlled closely.

The durability of tillite in a base depends on the composition of the rock and the stage of weathering. Fresh rock, i.e. blue tillite, is durable. For weathered rock crushing tests suffice if the rock does not contain more than about 30 per cent of components which are likely to decompose. The strength of such a rock should be determined in the 10% FACT and it is recommended that it should be at least 160 kN on the dry material when used for heavily trafficked roads, and it may be relaxed to 140 kN on the dry material for light to medium-trafficked roads, provided the wet test gives at least 70 per cent of the dry test result in all cases. If the rock contains more than 30 per cent of potentially decomposing constituents, i.e. erratics of acid and basic crystalline rocks or primary minerals, and if it is to be used in an environment where N is less than 5, the degree of decomposition of these

constituents must be taken into account according to what has been said about decomposing rocks.

Subbase, selected subgrade and fill: Highly weathered tillite is suitable if it satisfies the usual requirements as regards grading, plasticity, bearing capacity etc. Special care is required if the rock contains more than 30 per cent of potentially decomposing components when the same as had been said for base will apply.

Gravel wearing course: Highly weathered tillite can be used and has been used provided the rock does not contain too many unweathered erratics, especially quartzite. Only highly weathered tillite is a possible material for gravel wearing courses and it must be selected and treated according to the normal gravel road specifications.

Concrete aggregate: Fresh tillite, i.e. the 'blue' variety, makes an excellent coarse and fine aggregate for both concrete structures and concrete pavements. In structural concrete, weathered tillite, i.e. the first brown type, can also be used where the aggregate will be fully embedded in cement.

Group: Metalliferous rocks

These rocks were previously called 'Metallic Ores'.

Three natural road building materials have been grouped together here because of a high degree of uncertainty as to their correct placing in one of the seven preceeding groups. All they have in common is that they are mine waste, or surface or near-surface accumulations of ore. They are:

1) ironstone, the most important member;
2) magnetite; and
3) magnesite, the least important member.

They have been used for the following purposes in road construction:

TABLE 28
Use of metalliferous rocks in Southern African road construction

Rock	Surfacing aggregate	Base	Subbase	Selected subgrade and fill	Gravel wearing course	Concrete aggregate
Ironstone		+	+	+	+	+.
Magnetite		+	+	+		
Magnesite			+	+		

When weathering, these rocks are far more likely to disintegrate than to decompose. No durability problems have been experienced with any of them.

Ironstone
Ironstone is the most important member of the group. It has been used for all lower layers of pavements and locally as crushed stone for concrete structures in large parts of the north-eastern Cape Province and Bophuthatswana. The material is obtained from waste of the iron mines in the Sishen-Postmasburg area and occasionally from borrow pits. The source of the ironstone is the banded ironstone of the upper portion of the Dolomite Series and of the Timeball Hill Etage of the Pretoria Series, both Transvaal System. Ironstone has also been used around Thabazimbi, Transvaal, where it comes from the same geological strata as in the Cape Province but where it has only been used for base, subbase and as selected subgrade. All these materials belong basically to rather different types of rock, mostly shales and conglomerates, which have been strongly impregnated by haematite. Originally, they were most likely ferruginous sediments very similar to ferricrete whose durability as a road building material has largely been improved by metamorphism. The Rhodesian ironstones which have been used near Salisbury, Fort Victoria, Que Que and between Bindura and Shamva are much older geologically (Shamvaian Formation) but otherwise they are very similar to the South African occurrences. The Rhodesian material is obtained mostly from the disintegrated top portion of the rock.

Use in road construction
Ironstone is durable and, besides grading, its suitability is determined by its strength. The crushing strength of fresh ironstone is high and values of about 300 kN must be expected in the 10% FACT on the dry material. The strength should be in accordance with the usual specifications (**South African Bureau of Standards**, 1976 (Table 33). The rock causes problems in regard to the economy of its use because, being about one and a half times as dense as most other natural road building materials, it is very expensive to haul and is often rejected on these grounds.

Base: Ironstone crushed and natural, has been used for road bases in all the areas mentioned above. Since the rock mostly lacks natural binders, it is best stabilized with cement. The natural gravel often has to be screened to remove oversize aggregate.

Subbase: Only disintegrated ironstone has been used for this purpose in the same areas in which it has been used for base. Blending with soil may be necessary and in the Transvaal it has often been stabilized with cement. Many ironstones were originally shale and such rocks tend to disintegrate into rather flaky pieces which, because of the great strength of the rock, do not break easily under the roller and are therefore difficult to compact.

Selected subgrade and fill: Again ironstone, now highly disintegrated, has been used in the same areas as for subbase. If not mixed with clay, this material does not pose particular problems in regard to drainage but former shales are again difficult to compact.

Gravel wearing course: In Rhodesia, ironstone has been used occasionally for gravel wearing courses. Only highly disintegrated rock which is plastic lends itself to this purpose.

Concrete aggregate: In the north-east of the Cape Province, crushed ironstone has been used as concrete aggregate for culverts and bridges and no problems have been experienced.

Magnetite
Magnetite has only been used in the districts of Thabazimbi and Brits, Transvaal. The source of the material is certain iron-rich layers near the base of the norite of the Bushveld Igneous Complex, and the road building material is obtained mostly from borrow pits where the magnetite has been concentrated in the pebble marker. Since the pebble marker separates the underlying residual soil or highly decomposed norite, both of which contain much montmorillonite, from any type of transported soil on top, the material produced is likely to possess a highly plastic, fine component mixed with coarse-grained, hard magnetite. As a consequence, the grading of the material is unfavourable; there are too many coarse and fine particles but only low quantities of intermediate sizes and the material has only been used when nothing else has been available.

The mass of ironstone and magnetite as used in road construction is very similar and the cost of hauling magnetite is therefore also high.

Use in road construction
Occasionally, magnetite has been used for *base, subbase* or *selected subgrade* provided grading and plasticity could be kept under control. When used for base and subbase, the material has often been stabilized with either lime and slagment, or cement. There are no durability problems.

Magnesite
Magnesite, the magnesium carbonate $MgCO_3$, is a decomposition product of serpentine which in turn is an alteration product of olivine. This material has been used but very seldom, in the Kaap Valley area of the Eastern Transvaal where it has been obtained from mine dumps. Being a decomposition product of the ultrabasic rocks of the Jamestown Complex, mostly peridotite and pyroxenite which have often altered to such an extent that the original rock is hardly discernible, the actual material that has been used for road construction is of course not the pure magnesite, which is mined, but the waste of which magnesite is simply the major component. The material is usually a mixed aggregate which consists of the decomposition products of the various country rocks.

Use in road construction
Magnesitic material has been used for *subbase* and as *selected subgrade* at a few localities. When used for subbase, it has been stabilized with cement. Used in lightly trafficked roads only, nothing to its disadvantage is known.

CLASS: SOILS

It has already been said that it is difficult to find an undisputed distinction between 'rock' and 'soil'. All the road building materials discussed so far have been rock if by 'rock' is understood the consolidated, cemented, hard component of the earth's crust. It has been unavoidable, however, to include the different weathered stages of these rocks which means that states had to be discussed which are still rock according the the petrological definition but whose engineering properties have changed to those of soil, demanding the relevant testing and treatment.

Natural road building materials are of course not confined to rocks in some state of weathering; soils, whether residual or transported, also have to be included. What has been said about rocks whose engineering properties are those of soil applies vice versa to certain types of soil which have been cemented secondarily to such a strong material that their road building properties become those of rocks.

Group: Pedogenic materials

These soils which are very important natural road building materials in Southern Africa because of their virtually ubiquitous presence and the ease with which they can be obtained, form a group of road building materials equal in status to the eight preceding groups.

Pedogenic materials are soils which have been impregnated with, usually cemented and in certain cases partly or completely replaced by, some additional component. The original soil has consequently been transformed and a new one has been formed. This process is expressed in the names of the resulting materials which are composed of a term indicating the impregnating or cementing component and the ending '. . .crete' for 'created' or 'made of'.

The following members of the group have been used in Southern African road construction:

TABLE 29
Pedogenic materials used in Southern African road construction

Member of group	Impregnating material
Calcrete (or dolocrete)	Calcium carbonate (and dolomite)
Ferricrete	Iron hydroxide
Silcrete	Silica
Phoscrete	Phosphate

Laterite with hydrated aluminium sesquioxide as the major 'impregnating' material may be added to this group. Laterite is, however, very scarce on the subcontinent and it does not play a role in road construction. **Grant** (1974) has called soft aluminium- and iron-enriched materials 'laterite' which becomes 'ferricrete' when hardening in air and **Gidigasu** (1976) has described their engineering properties.

Impregnation starts with the precipitation of fine particles of the relevant material and in time these concentrate. Accretion of these particles leads first to soft, discrete nodules which consist mostly of the precipitated material that cements particles of the soil. These nodules seem to float in the host soil. Gradually, these nodules coalesce and a spongy, hard mass full of small cavities, the whole mass

having a honeycomb structure, is formed. The cavities are usually filled with remnants of the host soil although the cementing material may already have become quite hard. The filling of these cavities with the precipitate eventually results in rock-like hardpan which, when weathering, breaks down into boulders.

Hardpan, particularly in the case of calcrete, can also develop without passing through a nodular or honeycomb stage. It is then formed from an ever-increasing quantity of the precipitating material which has coalesced and replaced or uniformly pushed aside much or most of the host soil. The development of these materials can come to an end at any stage and not all stages need be passed through during the process of formation.

Since the precipitation of the material concerned is connected to its upward movement in the host soil, the strongest part of each succession lies on top, and therefore, whatever this strongest part is, the material becomes weaker from the engineering point of view if one moves down the profile (*Plate 25*). This reversal of the direction of change of the engineering quality within a profile is the important property which distinguishes the pedogenic materials from rocks although the hardpan has very much the properties of certain rocks: calcrete that of limestone or, if the carbonate is predominantly dolomite and the material should rightly be called 'dolcrete' of 'dolocrete', that of dolomite; the rather rare hardpan of ferricrete that of ironstone; and silcrete that of quartzite.

The described sequence of development can be observed most clearly in those pedogenic materials whose impregnating component is very different from the impregnated soil. This applies in particular to iron hydroxide and in most cases also to calcium carbonate and this development can therefore be observed best in ferricrete and calcrete. In the case of calcrete developing directly from, and directly on, limestone or dolomite, it may be difficult, however, always to distinguish clearly between the pedogenic material and the host soil if the latter is disintegrated limestone or dolomite.

Netterberg (1969, 1971), in his detailed study of Southern African calcretes, based his classification on this development: leaving aside calcareous soil as still being a host soil with a small precipitation of calcium carbonate which gives the soil a whitish hue, the lowest grade is *calcified soil* and this is followed by *powder calcrete*. In both these stages, the carbonate is extremely fine grained, normally cryptocrystalline, and there is only very little if any cementation. The main difference between these two stages is in the quantity of carbonate: this is less than 50 per cent in the calcified soil and more, mostly much more, than 50 per cent in powder calcrete, with strong replacement of the host soil as a consequence. Although there is little or no cementation in these two stages, it is conspicuous in the following four: *nodular calcrete, honeycomb calcrete, hardpan calcrete* and *boulder calcrete*. The distinction between nodular, honeycomb and hardpan calcrete is the degree to which the nodules have coalesced. Boulder calcrete is the result of weathering, being a combination of disintegration and solution which is very similar to the weathering of limestones (*Plate 26*). Thick weathering calcrete deposits may even house minor sinkholes as can be observed in the country around Bredasdorp in the Cape Province. During the process of enrichment, the particles of the host soil are either enveloped by the carbonate or chemically
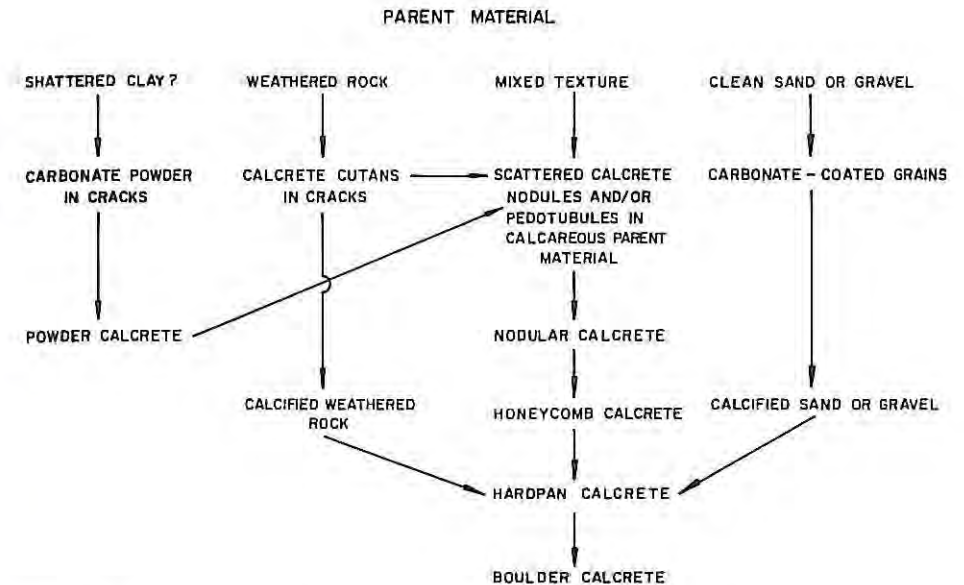
Figure 21: *Suggested sequence of calcrete development (after Netterburg, 1969, 1971)*

and/or physically replaced or pushed apart so that there are hardpan or boulder calcretes in which hardly any token of the original soil can be traced. The occurrence of the various types of calcrete also depends on the parent material and **Netterberg** (1969) has shown that this leads to various sequences of calcrete development (*Figure 21*).

Netterberg's classification of calcrete, which is of both geological and engineering significance, can be applied in a similar fashion, although with some reservations, to ferricrete and silcrete while phoscrete is different.

TABLE 30
Classification of pedogenic materials

Carbonate (after **Netterberg**)	Silica	Iron hydroxide
Calcareous soil	?	Ferruginous soil
Calcified soil	?	Ferruginized soil
Powder calcrete	?	Powder ferricrete
Nodular calcrete	Nodular silcrete	Nodular ferricrete
Honeycomb calcrete	Honeycomb silcrete	Honeycomb ferricrete
Hardpan calcrete	Hardpan silcrete	Hardpan ferricrete
Boulder calcrete	Boulder silcrete	Boulder ferricrete

Without adjustment, **Netterberg's** classification can be applied to ferricrete although the equivalent to powder calcrete, powder ferricrete, is rare. The reason for this similarity is probably that both precipitates, carbonate and iron hydroxide, have a composition very different from that of most soils, that they can therefore be recognized easily as additional constituents, and that they can exist in the soil without reacting with its components. Nodular and honeycomb ferricretes are more frequent as surface deposits than are the same stages of calcrete, in contrast to hardpan ferricrete which is quite scarce.

Silcrete is different in the loose stages but it is compatible with the scheme in the cemented stages where the honeycomb stage is the one occurring most frequently. If there were a siliceous or silicified soil or powder silcrete it could only be recognized by sophisticated laboratory investigations. These materials develop by the cementation of the soil by amorphous silica which first occurs in the soil as a gel. This gel solidifies to opal and later crystallizes into chalcedony, a modification of crystallized SiO_2, and still later into quartz (**Grant**, 1974). There is little certainty about the source of the silica which may be derived from diatom skeletons, decaying grass, decomposing minerals or the break-down of clay which is virtually always replaced where silicification takes place – hence there are no 'siliceous' or 'silicified' clays. In saline areas, the presence of sodium salts may act as a trigger for silcrete formation.

The silica gel and quartz grains in sandy soils grow by accretion and they eventually coalesce. Even if the 'droplets' of silical gel solidified without being bonded to some other component, they would only appear to be sand. Therefore, although siliceous or silicified soil and powder silcrete might exist theoretically, this could only be recognized by sophisticated means of analysis while the relevant material would appear sandy and would have to be considered as such for the purpose of road construction.

Total or partial replacement of calcrete by silica is not an exceptional occurrence. Total replacement leads of course to silcrete. Partial replacement, however, may allow the calcrete to retain most of its properties as a carbonate but its hardness may rise to as much as 6 on the Mohs' scale and its strength may exceed 180 kN on the dry material in the 10% FACT, which is equivalent to an ACV of about 22. Transitions between silcrete and ferricrete also exist with an intermediate stage which is rather similar to ironstone; more frequently, however, a layer of silcrete underlies ferricrete, particularly in a climatic environment with an N-value near 5.

Phoscrete differs from the other three members of the group. It is often calcrete which has been impregnated by phosphates derived from guano. The occurrence of phoscrete is therefore restricted mostly to coastal areas. A distinction must be made between phoscrete and phosphorite. Phoscrete develops by pedogenesis and the accumulation of phosphates is a secondary event in a pre-existing soil. Phosphorite, on the other hand, is of marine origin and develops mostly where decaying organisms but also decomposing rocks provide the phosphate which is then deposited with the other sediments, often in the form of oölites (**Tankard**, 1975). It appears that the phosphate deposits in the south-western Cape, especially along the coast and in the hinterland of Saldanha Bay, are of both types.

The regional distribution of calcrete and ferricrete is controlled by the avail-
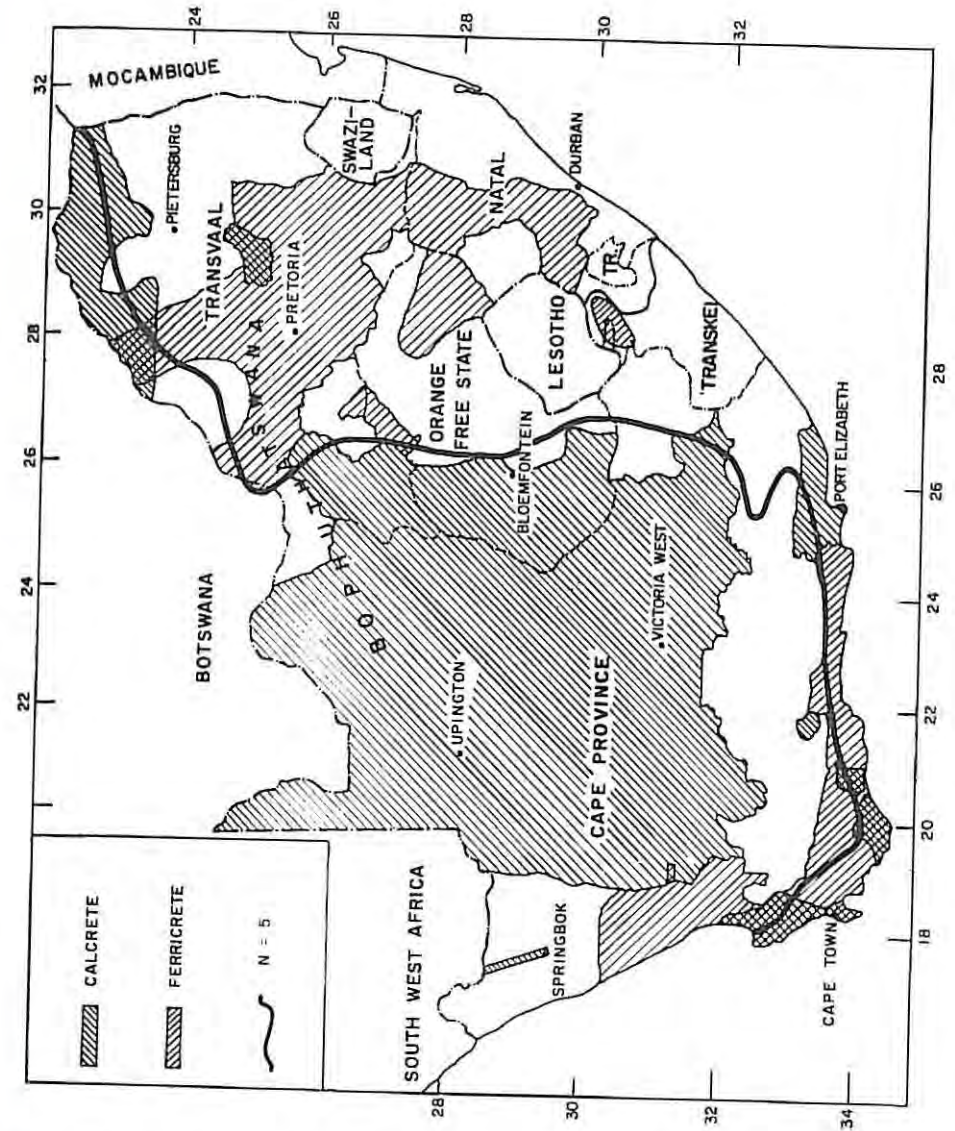
Figure 22: *The occurrence of road-worthy calcrete and ferricrete in relation to the contour N=5*

ability of carbonate and iron hydroxide. It is also controlled by climate which means that the relevant pedogenic material can only develop if its constituents are available in the correct climate. Particularly if one is considering the suitability for road construction, the important climatic factor is again N = 5: calcrete of road-making quality is found mostly where N is more than 5 and such ferricrete where N is less than 5. This is shown in Figure 22 and it applies equally to Rhodesia and South West Africa. This does not mean that calcrete or ferricrete never occur outside these climatically defined areas. Calcrete remnants from past climates occur in the Central Transvaal and at several places near the coast of the Cape Province as does fossil ferricrete near the west coast, particularly in the mist belt, and in areas in central South West Africa, e.g. near Otjiwarongo, and use has of course been made of these materials.

It is obvious that pedogenic materials require moisture in some form for their formation. Depending on the quantity of moisture available, or rather its supply mainly by rain, the climatic conditions of an area will be more favourable for the formation of either calcrete or ferricrete: silcrete does not appear to be so dependant on climate. The climatic equilibrium which favours the development of either calcrete or ferricrete is often quite delicate and minor variations may trigger the process of formation or bring it to an end (Weinert, 1976).

The development of calcrete is largely, although not solely, dependent on the evaporation of moisture which carries calcium carbonate in solution. Where there is not a sufficient supply of moisture, as for example under extreme desert conditions, calcrete will not form (Netterberg, 1971). Calcium and carbonate must of course be available and calcrete will therefore develop where calcium is either obtained from the country rock or imported by flowing water, while the carbonate can always be obtained from the air. Calcrete may therefore also occur on rocks which do not contain calcium themselves, e.g. sandstone. On the other hand, the available moisture must not be too plentiful because in such a case most of the calcium will stay in solution and be carried away. The best conditions for the formation of calcrete therefore occur in environments with an N-value of more than 5 and the conditions become unfavourable when N exceeds 50. In suitable areas, calcrete usually develops along perennial or intermittent rivers or streams, even if the flow is very steady as along the Omurambas and Oshanas in the northern parts of South West Africa. It is also found on the banks of pans and on rocks which contain sufficient calcium, such as carbonate rocks, but also basic crystalline rocks, argillaceous rocks and others when sufficient moisture is available periodically (Netterberg, 1971).

The formation of ferricrete requires the percolation of water through a soil and the presence of a temporary, fluctuating perched water table. Rain or other surface water must not evaporate too fast and the best climatic conditions for the formation of ferricrete are therefore encountered where N is less than 5. The percolating water mobilizes ferrous iron, mostly obtained from decomposing mafic minerals, and this ferrous iron is then carried downwards more or less to the base of such a perched water table. During this process the conversion to ferric iron takes place and the latter, being insoluble, is precipitated. Since precipitation occurs where the quantity of water in the soil decreases and where there is no import-

ant flow, i.e. near the base of such a perched water table, ferricrete is always deposited below some surface soil on planated surfaces, on gentle slopes of gully heads, on the sides of pans and near the junction of hill slopes and the pediment (Goudie, 1973; Partridge, 1975) and ferricrete exposed on the surface mostly indicates subsequent erosion and loss of surface cover.

The occurrence of silcrete is not linked so closely to climate although it is more likely to be found where N is more than 5.

Pedogenic materials have been used for the following purposes in road construction:

TABLE 31
Use of pedogenic materials in Southern African road construction

Material	Surfacing aggregate	Base	Subbase	Selected subgrade and fill	Gravel wearing course	Concrete aggregate
Calcrete	+	+	+	+	+	+
Ferricrete		+	+	+	+	
Phoscrete		+				
Silcrete	?	+	+	+	+	?

The widest use has been made of calcrete and, considering the whole of Southern Africa, calcrete is probably the road building material which has been and is being used in larger quantities than any other natural material. This is largely due to the occurrence of calcrete in the wide, sand-covered regions of the Northern Cape Province, Bophuthatswana, large parts of South West Africa, particularly in the north, the Western and Northern Transvaal, Botswana and parts of the south of Rhodesia, and to its frequent use for subbase, selected subgrade and fill, or for gravel wearing courses. In all these areas, calcrete is often the only hard road building material available. In the other parts of regions where N is more than 5, calcrete occurs as a surface cover and is consequently accessible and can mostly be worked easily, e.g. by ripping or bulldozing.

Ferricrete is not used as frequently. It occurs in an enviornment where the absence of a thick sand cover and the prevalence of more intensive rock weathering provide a wider choice between a number of suitable materials near most sites. Layers of ferricrete are seldom more than a metre thick which also reduces the frequency of use.

Phoscrete has only been used occasionally in the vicinity of Saldanha Bay.

Silcrete has been used in the west of Rhodesia, particularly in the Victoria Falls area. It is not certain, however, whether it has ever been used in the Republic of South Africa, e.g. in the South-Western Cape Province, since it may have been kept on records as 'quartzite' because of a misinterpretation of the nature of the material or because of the misconception of its older name 'surface quartzite'.

The precise description or denomination of pedogenic materials is occasionally

difficult because certain of the different cementing materials may be intermixed, either interspersed among each other or occurring in alternating layers of varying thickness. Silica in particular often occurs together with carbonates or iron hydroxides, and calcretes with a hardness of more than 3 or a crushing strength which approaches 200 kN have been found to be variably siliceous (Netterberg, 1971).

Use in road construction
All pedogenic materials have in common that they are secondarily cemented soils and that the cementations can produce materials of such a strength that a complete-ly cemented pedogenic material possesses the properties of solid rock in contrast to the uncemented material which is soil in every respect. This would also apply to weathering rocks where the thoroughly weathered layers call for treatment as a soil in contrast to the treatment demanded by the fresh rock. The difference be-tween conventional rocks and pedogenic materials, however, is that the latter pro-duce the highest degree of cementation and strength, and consequently that they possess their most rock-like properties at the top of the succession. Since this high-est degree of cementation need not be reached in the development of all pedogenic materials, it must be realized that, if the hard and strong layer is missing, it cannot normally be expected lower down the profile – the material becomes weaker with depth. Such weaker layers cannot of course be readily recognized in the case of sil-crete. This does not exclude the possibility of the superimposition of pedogenic materials from more than one cycle of formation. The formation of a calcrete for instance may have proceeded to the hardpan stage during a certain period, the hardpan even having been exposed to the surface for some time, before it was covered by another soil deposit, e.g. wind-blown sand. A new cycle of calcrete formation may now have derived much of its carbonate from the buried calcrete deposit without destroying it completely. This new cycle may have come to an end at any stage of development. In this case, an older, hard, strong pedogenic horizon may well be found below overlying softer pedogenic material, but within each cycle of formation the strongest material must be expected at the top of the rele-vant profile. Ferricrete and silcrete are not easily dissolved under a cover of soil. Superimposing successions will, however, also develop if the covering soil or the drainage pattern of the area provide for the import of ferrous iron and the precipit-ation of ferric iron or chemically active silica.

Pedogenic materials are soils and secondary impregnation or cementation is al-so a process which occurs under surface conditions. The host soil and the cement-ing material are therefore more or less in equilibrium with the surface conditions and, when weathering, these materials mainly disintegrate and, in the case of cal-crete, dissolve.

The degree of cementation is thus one factor which determines the suitability of pedogenic materials for road construction. Their properties are further determin-ed by the nature of the host soil which can be gravel, sand, clay or weathered rock. Generally, the impregnation of the soil has a beneficial effect on non-cohesive soils, which receive a natural binder, and on cohesive soils, whose plasticity is re-duced. This benefit is conferred particularly by calcrete and ferricrete, silici-

fication does of course not create a cohesive soil but may well reduce plasticity. With increasing impregnation, the components of the hoist soil are pushed aside and their relative quantity in the pedogenic material eventually decreases to such an extent that the quantity and quality of the host soil becomes negligible for most purposes of road construction. However, the quality of a pedogenic material which still contains an appreciable quantity of the host soil, i.e. one in which the nature of the host soil can be detected without trouble, must be assessed in the light of the properties of this soil in conjunction with the properties of the impreg-nating material.

The properties of the host soil are of particular importance in the uncemented stages of the nodular material. When selecting pedogenic materials, the nature of this host soil must therefore be assessed. This is quite easy as long as one deals with those stages which are still called 'soil' (see Table 30) and it is also easy in many occurrences of the nodular stage. The quantity of nodules may, however, be so plentiful that the host soil only fills the interstices between the nodules; this hap-pens particularly in pedogenic materials which are approaching the honeycomb stage. In this case, it becomes necessary to check the strength of the nodules and, especially when dealing with ferricrete, to determine the type of material which makes up the interior of the nodules.

Calcrete nodules are usually quite homogeneous in regard to the degree of ce-mentation and strength and they may consist of almost pure calcium carbonate or particles of the host soil evenly cemented with calcium carbonate. Taking the cli-matic environment of calcrete formation into account, the host soil as well as the cemented soil particles in the nodules are more likely to be sandy (sand or dis-integrated pieces of rock) than clayey but, since the latter is possible, the presence or absence of clay must be considered and its effect on the road structure must be ascertained.

Ferricrete nodules or pebbles consist mostly of a relatively strong, mainly dark to blackish-brown outer shell, which may be several millimetres thick, and a hard or soft but noticeably lighter-coloured material in the interior. This material in the centre of the nodules appears to be host soil which has been pushed together by the precipitation of the iron hydroxide and it can be observed that nodules with a sandy interior generally possess a greater overall strength than those whose in-terior contains clay (Van der Merwe, 1971). This difference in the strength of the interior is not due only to either sand or clay but also to the more intense pene-tration of the iron hydroxide between sand grains than between clay particles. If these nodules are used in a road layer, and they are used more frequently than any other stage of ferricrete development, they are inclined to break or at least to develop cracks in the outer, strong shell, particularly during compaction. These cracks then provide an access for water into the interior with obvious consequen-ces if this interior is composed of clay. Ferricrete which has cemented a sandy host soil is therefore more durable than ferricrete which has developed in clay, disinte-grated shale or decomposing rocks.

The host soil is of little if any relevance in hardpan or boulder silcrete which is as durable as high-silica rock wherever it occurs.

When a pedogenic material, especially ferricrete, which has formed in or on a

clayey material is excavated, great care must be taken not to excavate too deep. There is usually no well-defined contact between the pedogenic material and the clay and, especially in the case of ferricrete where even the colour contrast is of little help, there is always a danger of mixing clay into the construction material.

The plasticity of pedogenic materials depends only partly on the host soil. Obviously, if the host soil is clay, this will make the pedogenic material plastic while sand will not contribute to plasticity. Pedogenic materials, especially calcrete and ferricrete, however, always possess some plasticity of their own. This plasticity is caused by the fineness of the carbonate or iron hydroxide particles and probably also by a certain degree of porosity; the latter is particularly marked in calcretes which have an appreciable amount of diatom skeletons which are hollow silicate skeletons of microfossils (algae). The water in these pores, which has little effect on the properties of the material, is also expelled in the oven and thus adds to any quantity of water measured in the test. The result is that both the liquid limit (LL) and plastic limit (PL) tend to rise, the liquid limit more than the plastic limit, with a consequent increase in the plasticity index (PI). It must also be kept in mind that, depending on the method applied, the execution of the relevant tests can lead to recrystallization of some of the tested material with a consequent increase in porosity. One therefore has to decide which value obtained from Atterberg limit tests is the 'correct' one and it has been stated that calcrete and other pedogenic materials possess an 'apparent plasticity'. The shrinkage limit of soils appears to be always smaller than the plastic limit, i.e. the shrinkage index, which is PL-SL, is positive. This is however not the case for a number of calcretes and probably also ferricretes and Netterberg (1971) has therefore suggested that the true plasticity index of calcrete should be defined as the difference between the liquid limit and either the plastic limit or the shrinkage limit, whichever is higher. He has also stated that, at least for calcrete, more importance should be attached to the linear shrinkage for a number of reasons, amongst them that the result does not depend on two other tests as is the case with the determination of the plasticity index. However, there is little experience of the relationship between linear shrinkage and road performance in Southern Africa while the plasticity index has been used for a long time. Since the accurate determination of the plasticity index of calcrete is difficult and since calcretes with substandard plasticity have performed satisfactorily even in road bases, account has been taken of Netterberg's findings by relaxing the plasticity requirements of these materials slightly. Whether the same can be said about ferricrete is not yet quite certain but Portuguese authors (Da Silva et al, 1967; Meireles, 1967) have reported that such materials performed satisfactorily in lightly trafficked roads in an environment where N is about 2 even if their plasticity index was 24. The plasticity-reducing effect of calcium carbonate on clay minerals, montmorillonite in particular, can be expected to be greater than that of iron hydroxide. A frequent clay-type component of calcrete is the rather plastic but not expansive mineral attapulgite (Netterberg, 1971). Ferricrete nodules and pellets which contain clay become active if the clay can be released when the nodules and pellets break, particularly during construction (Van der Merwe, 1971).

In general, therefore, a pedogenic material developed in a sandy soil which contains quartz grains or pieces of any type of disintegrated rock (except members of the argillaceous group) makes a more durable and stronger road building material than one developed in clayey soils or rocks.

The durability of pedogenic materials is not as easily defined as that of rocks in which a certain stage of weathering must not have been exceeded. As long as loose, uncemented materials are used, the usual soil tests and specifications apply except that the plasticity demands can be relaxed. In the honeycomb, hardpan and boulder stages and for single nodules, strength tests are most revealing and the 10% FACT on dry and wet material is particularly recommended. The strength of hard and apparently intact calcrete in the 10% FACT varies between about 20 kN and more than 200 kN on the dry material and the acceptable decrease of strength of the wet material is about 60% in most cases. The strength of dry nodular and honeycomb ferricrete will hardly ever exceed 100 kN in the 10% FACT and most results are found to be between about 25 and 80 kN. Unfortunately, no information is available on the crushing strength of wet ferricrete but it can be expected to be considerably below that of the dry material. No strength test results on silcrete have become known but its crushing strength can be expected to be below 100 kN in the dry test if the material is very porous or shattered.

The difference in crushing strength between the various types of pedogenic material is largely a function of the strength of the impregnating material – carbonate, iron hydroxide or silica, silica being the strongest – and the quantity of this material present. Silica occasionally replaces the two other materials. Ferricrete is not readily replaced by silica and, where ferricrete and silcrete occur in the same profile, ferricrete is usually superimposed on silcrete. Replacement of calcrete by silcrete, however, occurs quite frequently and all calcretes having a dry strength of about 160 kN and more can be expected to have been replaced to a lesser or greater extent by silcrete.

The wet strength of pedogenic materials in the 10% FACT differs considerably from the dry strength. This is due to the rather high porosity and the frequent shattering of these surface materials. Since road construction involves the use of considerable quantities of water, in the case of most calcrete bases about ten per cent by volume, and since the materials in the lower layers of the pavement always retain a certain quantity of moisture, this decrease in strength in the moist state is of particular significance; it is therefore essential that strength tests on pedogenic materials be done on dry and wet material.

Some calcretes and ferricretes possess a property which makes them distinctly different from all other natural road building materials: after excavation they may resume the process of development from a loose to a cemented material. This means that some of them can improve in quality during service, a process which has become known as *self-cementation*. The underlying mechanisms appear to be different for the two materials.

Netterberg (1975) has stated that some of the carbonate in calcrete goes into solution during construction and also during rain, and that it is precipitated again during periods of drying. Repeated wetting and drying and the upward movement of the dissolved material, as happens during the development of calcrete under natural conditions, leads to the accumulation of the carbonate either on top of the

highest layer for which calcrete has been used or on the surface of a gravel road. Cementation of the rest of the relevant layer also occurs but is less pronounced.

Aitchison and Grant (1967) and Grant and Aitchison (1970) have suggested that ferricrete is subject to self-cementation if it still contains ferrous iron, Fe^{++}. Ferrous iron is readily soluble if the pH of the soil moisture is below 7 while ferric iron, the stage of higher oxidation, Fe^{+++}, is only soluble when the pH is below 4. Natural waters are, however, unlikely to have such a low pH and 'acid' natural waters mostly have a pH of hardly less than 6. Under such conditions the ferrous iron dissolves, is oxidized to the higher state and precipitated again. This precipitated ferric iron is deposited between the soil particles where it acts as a cement.

The stabilization of pedogenic materials, using either lime or cement, cannot always be done successfully on the basis of the usual rules: lime to reduce plasticity, cement to increase strength. Netterberg (1971) has shown that calcretes of low plasticity which contain tiny particles of amorphous silica, mostly diatoms or other microfossils, yield higher and more rapid strength with lime than with cement. The amorphous silica reacts quickly with the lime, probably forming in the lime the cementitious tobermorite gel. In fact, such a calcrete already contains the one important component required to achieve the intended pozzolanic effect. Calcrete which does not contain such tiny particles of amorphous silica or which has been partly replaced by coarser masses of silica must be stabilized with cement if a substantial and rapid strength increase is desired.

It is not known to what extent such tiny particles of amorphous silica occur in ferricrete and silcrete. They may be present, however, and the experience gained with calcrete could then be transferred to these materials as well. At this stage, however, nothing is known probably because in ferricrete the host soil remains the overriding component and the concentration of iron hydroxide hardly ever becomes so important quantitatively that the host soil virtually disappears, and because silcrete can only be recognized with certainty when nodules have formed or the honeycomb stage has been reached and the material is then already rather massive. The properties and performance of phoscrete are generally similar to those of calcrete.

Surfacing aggregate: Calcrete has been used as a surfacing aggregate in Southern Africa only on a very limited scale but this is not so certain for silcrete. Some of the aggregate recorded as quartzite may well be silcrete, particularly in minor roads in the nothern and central parts of the Cape Province, in the Cape Peninsula and between Karasburg and Nakop in South West Africa. The strength specification of the South African Bureau of Standards (1976) should apply to silcrete (Table 33).

Calcrete has been used for surfacing chips for lightly trafficked roads in Australia. The same could be done in Southern Africa if the calcrete is strong enough and not too porous, i.e. if it has a crushing strength of more than 180 kN on the dry material in the 10% FACT and provided that the wet test gives more than 65 per cent of the dry test result. There will not be many suitable calcretes and most of those which are suitable will be highly silicified. The same would apply to phoscrete. Ferricrete is unsuitable as a surfacing aggregate. Particularly in Angola, the Portuguese have, however, primed ferricrete bases successfully. These roads, which carry very little traffic (less than 100 vehicles per day) gave satisfactory service for about two years after which they received an overlay with aggregate of crushed rock (Meireles, 1967).

Base: All pedogenic materials have been used successfully for bases of light- to medium-traffic roads. The most suitable materials are obtained from the nodular stages, followed by the honeycomb types which are more difficult to work. Crushed calcrete and probably also crushed silcrete, in both the hardpan and boulder stages, have also been used. The properties of *silcrete* are so similar to either sand or quartzite, depending on the consistency and the degree of disintegration, that the material must receive the same treatment, and the specifications must be based on the same criteria, as for sand or high-silica rocks. Calcrete and ferricrete both differ from silcrete and from each other.

According to Netterberg (1971), *calcrete* nodules or crushed calcrete should have a dry crushing strength of at least 100 kN, possibly 110 kN, in the 10% FACT while the wet strength may perhaps be as low as 50 per cent of the dry strength when used in roads with up to medium traffic. The dry strength may be relaxed to about 80 kN if the expected traffic is less than 1 000 vehicles per day. The plasticity requirements for calcrete, particularly if nodular calcrete is used, can be relaxed to a plasticity index of 8 for heavily trafficked roads and to as much as 15 for light traffic, with adjustments for intermediate conditions. Although roads with a natural calcrete base which carry low traffic have performed satisfactorily, the value of 15 for plasticity is not acceptable to all road authorities some of which do not accept calcrete with a plasticity of more than 10 even for the bases of low-traffic roads. If the host soil is disintegrated argillaceous rock or clay, greater care is required in the selection of calcrete than when this soil is sandy. The relatively high percentage of fines in natural or crushed calcrete is a normal property of this material. As a result, the requirements for grading need not be as strict as for other materials particularly because these fines do not constitute a danger to a road structure.

Crushed *phoscrete* with a soil binder has been used successfully for bases in the Saldanha Bay area.

Virtually all *ferricrete* which is used for bases is obtained from the nodular and honeycomb stages. In spite of the low crushing strength, less than 100 kN, ferricrete has given very satisfactory service mostly in areas where N is less than 5. This property, which seems to contradict all experience with other materials, is possibly the result of self-cementation and the irregular shape of the ferricrete nodules. However, this only applies to ferricrete whose host soil is sand. The assessment of the nature of the host soil is therefore of prime importance when dealing with ferricrete. Being poorly graded in their natural condition, weak nodular and excavated honeycomb ferricrete are subject to a high degree of degradation during ripping, hauling, stockpiling and rolling. As long as the host soil is sand or sandy, but not silty or clayey, this degradation may improve the grading and the sand component maintains the strength. The iron components, particularly those in the softer condition, when released from the broken nodules, act as a natural binder. Nodules and pellets which are not broken provide very good interlock of the coarse particles because of their normally quite irregular shape. The result is a sand- to gravel-

sized material with sufficient fines, many of which may even be self-cementing, and which can be compacted easily. To obtain more strength, such a material can be stabilized with cement. Ferricrete, whose host soil is silty or clayey, does not possess these advantages. The silt or clay components are set free during handling, the material becomes rather plastic, the compression strength decreases noticeably and the material must be treated like a ferruginous clay soil which should possibly be avoided for use in the base especially if the road carries more than 200 vehicles per day. The iron component has, however, a beneficial effect on the clayey host soil and, in such lightly trafficked roads, ferricrete with a plasticity index of as much as 21 has been used successfully in Angola (**Meireles**, 1967). This Portuguese observation obviously indicates that the plasticity requirements may also be relaxed for ferricrete (even if the above reported case is a very extreme one) perhaps to a degree similar to that applied to calcrete.

Sandy ferricrete has given excellent service under thin bituminous surfacings and it could be considered as an alternative to crushed stone bases even in major structures, as has been shown at Salisbury airport (**Van der Merwe**, 1971).

Subbase, selected subgrade and fill: All pedogenic materials are suitable for the lower layers of a pavement and particular use has been made of the less-cemented stages of development. When it is stated that calcrete may be the road building material of which the largest quantities have been used in Southern Africa as a whole, this is largely due to its frequent use for these lower layers in all areas where N is more than 5. The host soil is still important when deciding on how the materials are to be treated. The specifications for the plasticity of calcrete can be relaxed and a plasticity index of 17 has been suggested for all classes of roads but a still higher value may prove acceptable especially for the subbase of low-traffic roads. In general, there is little agreement about this point among the different road authorities and in practice acceptance and rejection are largely based on local experience. The plasticity requirements for ferricrete could probably be relaxed in a similar fashion, at least provided the host material is sand, but greater precaution is definitely required when the host material is clayey.

Chemical stabilization of pedogenic materials must generally be carried out according to the usual rule of lime to reduce plasticity or cement to increase strength, but the presence of amorphous silica, such as diatom skeletons in calcrete, may cause a rapid gain in strength even when the material has been stabilized with lime.

Calcrete, particularly calcified soil or powder calcrete, may require special precautions when used for high fill. Seepage must be avoided because the seeping water, which is unlikely to have been calcium-saturated originally, may dissolve particles of the carbonate, particularly if the surface area of these particles is large as is the case when calcified soil or powder calcrete are used. The water leaving the fill would then be saturated with dissolved calcium carbonate and piping may occur. No such cases have been observed so far in Southern Africa, however, and this postulate should only serve as a warning in the light of the increasing number of high embankments.

Phoscrete has never been used in the lower layers of pavements.

Gravel wearing courses: Calcrete, ferricrete frequently and silcrete only in the

Victoria Falls area have been used for gravel wearing courses. Calcrete and ferricrete are suitable materials which, because of their self-cementing properties, develop a rather smooth riding surface, a crust, occasionally called a 'blad' (*Plate 27*), if careful watering and rolling are applied repeatedly. This crust does not require much maintenance but if it breaks, potholing and rutting rapidly occur. Such roads are not prone to corrugate. Since a gravel wearing course must be plastic to a certain degree, pedogenic materials whose host soil is clayey are very suitable. Very suitable material is also obtained from the nodular stages in which case, besides grading, the aggregate strength, i.e. the crushing strength of the nodules, is the most important single factor in selection. The aggregate pliers test or its simplified version, the aggregate fingers test (see Chapter 7), have been developed by **Netterberg** (1971, 1978) mainly for determining the suitability and durability of nodular calcrete for gravel wearing courses. A suitable and durable nodular calcrete should have an aggregate pliers value of more than 20 per cent or an aggregate fingers value of more than 60 per cent. These percentages refer to the quantity of nodules which cannot be broken by an ordinary pair of 180 mm pliers or between thumb and forefinger. These tests should also be applicable to ferricrete. In nodular pedogenic materials in which the fraction passing the 0,425 mm sieve is greater than 45 per cent, the strength of the nodules is overridden by the soil constants, liquid limit, plasticity index, linear shrinkage etc., of the fine material in which the nodules only float.

Silcrete for gravel wearing courses performs like a gravelly sand and it must be selected and treated accordingly.

Phoscrete has never been used for gravel wearing courses.

Concrete aggregate: Silcrete has been used for concrete aggregate in the Rundu area of South West Africa. It may, however, also have been used in other areas but, since the material has probably been recorded as 'quartzite', nothing definite can be said about its performance. Silcretes may contain amorphous silica (**Aitchison** and **Grant**, 1967; **Mountain**, 1967) and their use as an aggregate especially with high-alkali cement may therefore be likely to cause excessive cracking of the concrete.

Calcrete has apparently been used unintentionally for a bridge over the Omuramba Ovambo on the road from Peret to Andoni in the north of South West Africa. The performance of the material in the concrete appears to have been satisfacory.

Both materials, silcrete and calcrete, should make suitable concrete aggregate although their use as fine aggregate may pose problems with certain cements because of the presence of amorphous silica. The strength of calcrete and silcrete if used as a coarse aggregate in structural concrete may be as little as 70 kN for the dry material in the 10% FACT, although this does not meet the approval of all authorities some of whom prefer not to accept aggregate of a strength of less than 110 kN. A strength of 110 kN in the 10% FACT on the dry material is normally specified for concrete aggregate which is subject to abrasion. This strength value appears somewhat low if the aggregate is calcrete in which case a strength of at least 140 kN or possibly 160 kN would be more appropriate. If the aggregate is hardpan silcrete, however, 110 kN will be acceptable. In general, reference should

be made to the specification 'Aggregate from natural sources', issued by the **South African Bureau of Standards** (1976). The wet test should give at least the usual 75 per cent of the dry test results on silcrete while a relaxation to 60 per cent would be acceptable for calcrete.

The same criteria as for calcrete probably apply to *phoscrete*, while *ferricrete* is unsuitable as a concrete aggregate. Phoscrete has, however, never been used as a concrete aggregate.

Group: Soils

'Soil' is here understood to be the unconsolidated material which covers most of the surface of the earth, which is the product of rock weathering and which may or may not have been transported to some place other than that of its origin. There are consequently two different types of soil:

a) *Residual soil* which is the unconsolidated material, gravel, sand, clay or mixtures of these which has developed solely from the weathering of the underlying bedrock, grading gradually into the bedrock as one moves down the profile.

b) *Transported soil* which is the unconsolidated material deposited after transport by wind, water, gravity or ice, the latter hardly being applicable to Southern Africa (**Sparrow**, 1974), or also by organisms, particularly animals, during which process the material is often but not always subjected to sorting. Transported materials can also be described as gravelly, sandy, clayey or any transition between these.

Road building properties
Most of the road building properties of *residual soils* have already been discussed in conjunction with the parent rock. Basically these properties depend on the type of parent rock and the mode of weathering, and both determine the resulting residual soil. If these soils contain much montmorillonite as is the case with decomposed basic crystalline rocks in an environment where N is less than 5 and with many of the disintegrated argillaceous rocks of Karoo age and younger, they are of hardly any use as natural road building materials.

All other residual soils can be and have been used successfully for the lower layers of pavements and for gravel wearing courses. Most of these soils, provided they were devoid of excessive montmorillonite, have been used for the mechanical stabilization of other materials, especially to adjust the grading. Kaolinite clay, if not too plentiful, i.e. not exceeding 20 per cent in most cases, is a valuable natural binder because it adds cohesion to the material and makes compaction easy and efficient. Where natural binders are not available or where they are unsuitable for other reasons, bitumen and tar may be used for mechanical stabilization. This method of stabilization has not often been applied to residual soils.

Chemical stabilization of residual soils is achieved by using either cement or lime depending on the type of soil and the intended purpose.

Cement must be used as a binder for non-cohesive soils, i.e. gravel and sand, to obtain strength. Sand-size residues on crystalline rocks are usually rather angular and therefore develop some strength by means of interlock. When the sand is residual from crystalline rocks, particularly the acid types, in an environment which favours decomposition, i.e. where N is less than 5, the natural clay content often provides a binder which, in conjunction with the interlock of the angular, still unweathered, sand-size minerals, supplies sufficient strength especially in low-traffic roads. Sandy and gravelly materials derived from sedimentary rocks, particularly from members of the arenaceous group, are often wind- or water-worn and rounded as a result of original transport before deposition and lithification. Such materials usually need to be stabilized with cement when they are used for the base and in many cases also for the subbase although in the latter case the addition of a soil binder may suffice, especially in low- to medium traffic roads.

Lime is required for the stabilization of cohesive residual soils to reduce plasticity. Lime is therefore used for highly decomposed rocks and the relevant residual soils where N is less than 5, and for the residual soil on argillaceous rocks and many diamictites, e.g. many Dwyka tillites, in any environment. Such materials are of course never used for a layer higher than subbase and mostly only from selected subgrade downwards. The effect of lime is complex and consists of ion exchange and pozzolanic action with amorphous silica which is often present in the gel form, particularly in soils which are derived from decomposing crystalline rocks and in which the relics of primary minerals are still in a state of decomposition. Certain soils of this type, particularly those deriving from basic crystalline rocks, argillaceous rocks and certain diamictites, and particularly where N is less than 5, tend to absorb so much lime that it is advisable to determine the initial consumption of lime (ICL) (**Clauss** and **Loudon**, 1971).

For the purpose of road engineering, it is sufficient to distinguish between the clay minerals kaolinite and montmorillonite, the latter being a member of the smectite group of clay minerals. The mineral attapulgite, which is also considerably plastic, is not really a clay mineral because it possesses a chain and not a sheet lattice. The principal difference between kaolinite and montmorillonite as far as road engineering is concerned, is their reaction with water. Both can absorb water in, and release it from, their crystal lattice. This makes them swell when wetted and shrink when dried. The quantity of water which can enter the lattice of kaolinite, however, is small and its effect on expansion or shrinkage is negligible for the purpose of road construction. This is not the case with montmorillonite which can expand to several times its 'dry' volume until it changes into a gel (**Grim**, 1968). The geological and climatic conditions of Southern Africa favour the formation of montmorillonitic clays and it is advisable to expect montmorillonite in every clay soil until the contrary has been ascertained. It may serve as a guide that montmorillonite can be expected to predominate in soils residual from basic crystalline rocks in areas where N is less than 5 and in soils residual from argillaceous rocks and diamictites of Karoo age and younger in any climatic environment. The indication obtained from the Atterberg limits, however, must be interpreted with caution because, although montmorillonite is much more plastic than kaolinite, admixtures of other components, e.g. sand or other unweathered residues, and the intermixture of the two clay minerals affects the results very much. This effect of other, additional materials is of practical importance when montmorillonitic soil has to be used for subgrade and fill; adding sand to such a soil has an almost linear

effect on the reduction of plasticity, e.g. adding 50 per cent sand by volume also reduces the plasticity index by about half. This procedure, which has been used on several occasions, is actually only the replacement of some of the plastic material by non-plastic material; a reduction in plasticity is the result.

Halloysite, a member of the kaolinite group of clay minerals, has the particular property that its clay platelets roll up into tiny tubes or cylinders which enables these clays to hold large quantities of water. This water is not chemically combined and it does not affect the mechanical properties of such a soil. Halloysite soils are marked by their low maximum dry density which is always less than 1 600 kg/m³ and in most cases even less than 1 500 kg/m³, and by their high optimum moisture content which is often more than 50 per cent. The mineralogy of halloysite has been described by **Grim** (1968) and **Dimanche** *et al* (1974) and its engineering properties have been discussed by authors such as **Dixon** (1963), **Wesley** (1973), **Kiek** (1974) and, as a result of the Speciality Session on lateritic soils during the 7th International Conference on Soil Mechanics and Foundation Engineering held in 1969 in Mexico City, compiled by **Moh** (1969). Halloysite soils are rare in Southern Africa; they are restricted to areas where N is at least less than 2 and in most cases even less than 1. Their use in road construction is restricted to fill and compaction is usually difficult, requiring a large number of passes although more than 100 per cent Modified AASHTO density can eventually be obtained. Fills built with such soils have not been known to cause problems.

Strictly speaking, nearly every topsoil is *transported soil* and, if it occurs on a complete weathering profile, it is separated from the residual soil by the virtually ubiquitous 'pebble marker'. Most of this material in Southern Africa is, however, only a thin deposit of colluvial or biotic soil which is of no importance in road construction because it is either removed or mixed with the underlying residual soil.

Masses of rock and soil which have moved downhill by gravitational force, either slowly as creep or spontaneously as a landslide, and which may contain blocks of a mass of many tons, are known as *talus* or *hillwash*. In Southern Africa, talus and hillwash are generally distinguished by the prominent grain size of the material: talus is composed of gravel and larger sizes without an upper limit, and hillwash is sand-size and finer. Talus usually occurs on the steeper part of a slope next to a cliff or some other steep outcrop of more or less bare rock, and hillwash where the gradient becomes more gentle. This type of colluvium is very unstable and can attain considerable thickness. The greatest care is therefore required if it is to be passed in a cutting or has to support a pavement (*Plate 28*). On the other hand, colluvial deposits can provide large, easily workable and easily accessible sources of material for selected subgrade and subbase.

Vast areas of the subcontinent, particularly where N is more than 5, are covered by aeolian deposits most of which are known as *Kalahari sand*. Large outliers of such deposits even occur in areas where N is less than 5. In addition to these continental accumulations, another type of aeolian deposit occurs in the occasionally wide dune belts along the coasts.

During the last few million years, the subcontinent has been subject to erosion and removal of material with the result that *alluvial deposits* have only developed locally. There are only a few rivers whose course is sufficiently long to encounter conditions where the load of debris can be deposited; most Southern African rivers reach the sea along a bed with such a gradient that they erode all the way from the source to the mouth.

Transported soils are of little importance in Southern African road construction. Wind-blown sands, Kalahari and coastal dune sand, have been used in gap-graded bituminous mixtures, in the lower layers of pavements, especially for subgrade, and quite frequently for gravel wearing courses. Since they possess no or only very little natural binder, they always require cement stabilization if used for a layer other than lower subgrade in a road structure. When used for a gravel wearing course, they are usually dusty and notorious for fast corrugation. Certain chemicals such as sulphite lye (**Fossberg**, 1966) have been applied to them with some success for the control of dust and corrugation; such chemical agents are, however, very weather-dependent and need to be reapplied frequently. Locally, wind-blown sands have been used, stabilized with cement or bitumen, for the base of roads which carry light traffic and in Rhodesia they have also been used locally as fine aggregate for bituminous surfacings. Their most important use, however, is perhaps for the adjustment of faulty grading.

The only alluvial deposits which have gained some regional importance as road building materials in the Republic of South Africa are the diamondiferous gravels of the Vaal basin, especially in the south-western Transvaal. These gravels are badly graded and contain large pebbles and even boulders with varying quantities of fine material including clay. They have been used in all layers of the pavement, stabilized mostly with cement but also with lime (if demanded by plasticity) when used for the base, and in the natural state for other layers. The large quantity of well-rounded pebbles and boulders has meant that they often only added bulk to the layer concerned so that they had to be blended with other soil. If sufficiently plastic and provided the size of the gravel permitted, they have also been used for gravel wearing courses.

River gravel deposits, which always provide rounded, water-worn aggregate, have been used in the coastal areas near the mouth of the Orange River and near river mouths from about St. Helena Bay to Mozambique. They pose very similar problems in regard to grading as do the diamondiferous gravels although they can seldom be called 'clayey'. Natural binder must therefore be sought away from the river or stream beds. These materials have been used for all purposes of road construction, always crushed for surfacing and concrete aggregate, either crushed or natural for the base, and always natural for the other layers and gravel wearing courses.

River sand has been used at many places as fine aggregate for bituminous surfacings and concrete.

Material deposited by rivers goes through a process of sorting according to the mass, and to a certain degree also the shape, of the grains. As a result the deposited material gets finer with increasing distance of transport and it eventually becomes clay when the speed of flow has decreased sufficiently. Where the water

flows fast, only the heaviest and least weatherable rocks and minerals are deposited and highly siliceous materials are more likely to be concentrated in such deposits than others, despite the observation by **Smalley** (1966) and **Moss** (1973) that such highly siliceous material, quartz and chert in particular, is quite brittle and tends to fracture on impact. This physical break-down does not, however, change the mineralogical properties of a quartz or other siliceous material, but the other frequently occurring rock-forming minerals decompose in the presence of water at temperatures normally encountered in Southern Africa. During this process they eventually change into clay minerals which are transported over a longer distance than the more compact debris until they are deposited as alluvial clays in slowly flowing or stagnant water.

River gravels and sands that are suitable for road building are therefore composed mainly, and often exclusively, of materials which belong to the group of high-silica rocks, quartzite, hornfels and vein quartz. In the diamondiferous gravels, chert is the major component and in addition there are jasper, agate, banded ironstone and other, highly siliceous rarer types of rock and mineral. The properties and potential road performance of these alluvial materials are therefore predominantly those of the group of high-silica rocks. In the short rivers on the coast between about Mossel Bay and Cape St. Francis, quartzitic sandstone, a member of the group of arenacous rocks, has been recovered, crushed and used for surfacing and concrete aggregate. This does not exclude the occasional occurrence of pebbles and boulders of other rocks if a river or stream is flowing, or has just flowed over any other type of country rock. The gravels of the Vaal River for instance contain large quantities of andesite at some places. Many other river gravels in the Transvaal contain norite or granite in their upper reaches and more such examples could be mentioned. In such cases the road building properties of such gravels are of course those of the groups to which the constituents of these gravels belong.

In certain areas of South West Africa quartz pebbles have accumulated in workable quantities. These pebbles have been derived from the weathering of rocks of the Khomas Series of the Damara System, often being the residue of disintegrated mica schist, and certain Karoo sediments. The quartz pebbles cover the surface in large areas of the central parts of South West Africa and they are often concentrated in the streets between sand dunes in the east. Quartz gravels, natural or screened, have proved to be excellent materials for the bases of all types of road, including freeways. Since the deposits consist almost exclusively of quartz pebbles, their road construction properties are of course those of the group of high-silica rocks. Care must be exercised, however, where they have been derived from mica schist especially when the mica is muscovite. The presence of this type of mica causes problems with the compaction of such soils (**Tubey**, 1961; **Tubey** and **Bulman,** 1964) and with the durability of compacted pavement layers and, if contained in the fine aggregate, with the workability, compressive strength and drying shrinkage of concrete (**Müller**, 1971). Nevertheless, the stabilization of micaceous structural layers of a road with cement dampens the spring action of muscovite to some extent (see Chapter 10).

Records of road building materials often indicate that '*local soil*' has been used mainly to adjust the grading or to add a natural binder to non-cohesive materials. There is no way of defining the composition of 'local soil' which could consist of any type of soil or weathered rock and which could be residual or transported.

SUMMARY
GENERAL

The use of the different types of natural road building material in Southern Africa is summarized in Table 32.

In this table, the individual types of rock are shown as members of the relevant groups. Each group of natural road building materials requires the same treatment, and the same precautions must be taken when any one of its members is used for some purpose in road construction. Each group is defined by the presence or absence of quartz, the minerals associated with quartz and the type of weathering resulting from this mineral assemblage in a given environment.

This grouping makes it unnecessary for the road engineer to decide whether a material is for instance a diabase, dolerite or norite, or a quartzite or vein quartz; it is quite sufficient to recognize the material as a member of one of the groups and it would even suffice if the engineer described it accordingly. All he needs to know and recognize to do this is one mineral, quartz, and he must then apply this knowledge in conjunction with the intended mode of road use in a known environment.

To the geologist, the recognition of minerals and specific types of rock is commonplace. This grouping enables him, however, to connect a set of road building properties with groups of materials whose individual members may be rather different according to the traditional genetic classification of rocks and soils.

In short, this grouping means that the distinction between, and recognition of, more than 40 different natural road building materials can be reduced to nine, or to ten if the unconsolidated soils are included.

When natural road building materials are being prospected for, materials for the lower layers should be located as near to the site as possible although, particularly for base material, quality is more important than hauling distance. Haulage of surfacing stone over several hundreds of kilometres has always been acceptable because of the high quality requirements. The following general rules may serve as a guide for the selection of natural road building materials.

SELECTION
A: Mineral composition of the material

1) *The more quartz a rock contains the less it will decompose and the more durable it will be.*

2) Igneous rocks, most metamorphic rocks (crystalline rocks) and some types of diamictite may either disintegrate or decompose, depending on the environmental conditions, in a particular climate (N-value) (Compare A(1)).

3) Disintegration is the predominant type of weathering of all other groups of rocks and the road building quality of such a rock depends mostly on its crushing strength and mineral composition.

TABLE 32
Natural road building materials used for different purposes of road construction in Southern Africa
(see also Appendix 7)

Material		Purpose					
Group	Rock	Surfacing aggregate	Base	Subbase	Selected subgrade and fill	Gravel wearing course	Concrete aggregate
Basic crystalline rocks	Amphibolite	+	+				+
	Andesite	+	+	+	+	+	+
	Anorthosite		+				+
	Basalt	+	+	+	+	+	+
	Diabase	+	+	+	+	+	
	Diorite	+		+	+	+	+
	Dolerite	+	+	+	+	+	+
	Gabbro	+	(+)	(+)	(+)	(+)	+
	Greenschist	+	+	+	+	+	+
	Norite		+	+	+	+	+
	Peridotite		+	+	+	+	
	Phonolite	+					+
	Serpentinite	+	+	+	+	+	+
Acid crystalline rocks	Felsite	+	+	+	+	+	+
	Gneiss	?	+	?	?	?	+
	Granite	+	+	+	+	+	+
	Pegmatite		+	+	+		
	Rhyolite	+					+
	Syenite	+	+				+
High-silica rocks	Chert		+	+	+	+	
	Hornfels	+	+				+
	Quartzite	+	+	+	+	+	+
	Vein quartz	+	+	+	+	+	+
Arenaceous rocks	Arkose	+	+	+	+	+	+
	Conglomerate		+	+	+		
	Gritstone		+	+	+	+	
	Mica schist			+	+	+	
	Sandstone	+	+	+	+	+	+
Argillaceous rocks	Shale, slate, mudstone		+	+	+	+	
	Sericite schist, phyllite		+	+	+		
Carbonate rocks	Dolomite	+	+	+	+	+	+
	Limestone		+	+	+	+	+
	Marble	+	+	+	+	+	+
Diamictites	Tillite	+	+	+	+	+	+
Metalliferous rocks	Ironstone		+	+	+	+	+
	Magnesite			+	+		
	Magnetite		+	+	+		
Pedogenic materials	Calcrete	+	+	+	+	+	+
	Ferricrete		+	+	+	+	
	Phoscrete		+				
	Silcrete		+	+	+	+	?

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4) The use of rocks containing easily detectable quantities of olivine, serpentine, muscovite, pyrites and marcasite must be considered with caution. Some rocks containing nepheline, particularly when the mineral occurs in basaltic rocks, may disintegrate quickly under the influence of the atmosphere ('sunburn') (see Chapter 10).

5) Kaolinite is a very suitable secondary mineral in natural pavement materials as long as its quantity does not affect the plasticity and strength requirements. Large quantities (more than about 10 per cent) of expansive minerals, especially montmorillonite, however, are always dangerous.

6) The quantity of secondary minerals in rocks that can decompose must always be in accordance with the N-value (Figure 11).

7) The quality of pedogenic materials depends both on the properties of the original host soil, e.g. its plasticity, and the stage of development of the material. The more sandy the host soil, the better the quality of a pedogenic material generally is.

8) The more heterogeneous the mineral composition of a rock is, especially if quartz is present in appreciable quantities, the less the rock will polish in road surfaces.

9) Any material intended for use in concrete roads or for structural concrete must be considered along the same lines as used in concrete technology.

B: Structure and texture of the material

1) A fine-grained rock will generally be more durable and stronger than a coarse-grained one.

2) A fine-grained rock will disintegrate less readily than a coarse-grained one.

3) A massive rock will disintegrate less readily than a laminated or schistose one.

4) A massive rock will be more suitable for crushing than a laminated or schistose one.

5) Rocks which have smooth surfaces after crushing, e.g. quartzite or vein quartz, give a weaker bond with bitumen than holocrysalline ones with rough surfaces.

APPENDIX 5: *A guide to the recognition of rocks*

A guide to the application of the 'rule of quartz' is given in tabular form on pp. 210–215. This guide is based on the assumption that the user is sufficiently familiar with the appearance of quartz and opal (amorphous silica) in rocks to recognize them macroscopically, i.e. with the naked eye or a hand lens (magnification 8 to 10 times).

The inspection of the sample must always be done on a freshly crushed face and the following equipment is the minimum required: a geological hammer, a hand lens and a steel needle or pocket knife; the availability of a bottle with diluted hydrochloric acid (HCl) is an advantage.

To read the guide, begin with the left-hand column and proceed column by column to the right. Every item must be considered and the reader must keep with-

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Quartz, Opal	Texture and mineral composition	Colour and lustre	Matrix	Fracture face	Intermixed other components	Additional physical characteristics	Group	Possible rock type	Remarks
All quartz or opal (amorphous silica)	Glassy to vitreous, very dense, uniform	Mostly white, but also many other colours, or colourless; shiny or glossy lustre		Smooth, concoidal	None	Very hard, cannot be scratched with needle or pocket knife	High-silica rock	Vein quartz Chert Hornfels (Obsidian)	Vein quartz occasionally contains galena (PbS)
	Dense	Mostly dark shades of grey		Smooth to slightly rough				Hornfels	
	Granular with grains of varying sizes	Many but shades of grey predominant; lustre shiny or glossy	Siliceous	Smooth, breaks equally through grains and matrix				Quartzite	Occasionally contains sulphide minerals, e.g. pyrite (FeS_2)
				May be smooth or rough depending on strength of cementing matrix	Variable, depending on host material	Cannot be scratched with needle or pocket knife; voids (honeycomb)	Pedogenic material	Silcrete	Distinction from quartzite often very difficult: seek expert advice
		Many but shades of grey predominant; lustre dull but individual grains may flicker in sunlight		Breaks run through matrix and the unbroken (sand) grains protrude: fracture face feels rough (sandpaper)	Occasionally single minerals, especially feldspar, and rock fragments	Cannot be scratched with needle or pocket knife but single sand grains may be removed in this way	Arenaceous rock	(Quartzitic) sandstone, grit conglomerate	
Mostly quartz or opal (> 50%)	Granular, mostly sand grains	Many colours, lustre dull	Calcareous, ferruginous or clayey	Very rough, feels like sandpaper; grains can be removed with needle or pocket knife	Occasionally feldspar; very rarely other minerals	Matrix can be scratched with needle or pocket knife; calcareous matrix 'boils' in hydrochloric acid (HCl)		Sandstone Gritstone Conglomerate Arkose (if containing Feldspar)	May contain mica or sulphide minerals, especially pyrite (FeS_2)
	Granular; alternating thin sheets of quartz and mica	Shades of grey, minerals flicker in sunlight	None	Smooth parallel with bedding, and rough perpendicular to beds	Occasionally minerals other than quartz and mica	Breaks into platy pieces. Scratched easily with needle or pocket knife on bedding planes		Mica schist	
	Granular, grains of various sorts and sizes	Mostly shades of grey; lustre dull	Clayey, rarely siliceous	Rough, feels like sandpaper; grains can be removed with needle or pocket knife	Contains angular to sub-angular fragments of all sorts of minerals and rocks	Strength may vary considerably. Resistance to scratching variable	Diamictite	Greywacke Tillite (Volcanic tuff) (Volcanic breccia)	Recognition of these rocks often difficult, use should be made of expert advice
	Dense to vitreous fine granular	Red to brown, generally dull but grains flicker in sunlight	Ferruginous	Smooth to rough: when rough it feels like fine sandpaper	None	Very heavy	Metalliferous rock	Ironstone	Occasionally shades of colour vary in fine layers (banded ironstone)
Quartz or opal prominent (> 10%, < 50%)	Vitreous, quartz as single crystals or as clusters may float in vitreous matrix; often signs of flow or turbulence	Mostly shades of red from light to dark but other colours as well; lustre dull to slightly shiny	Dense, rather uniform mass	Rather smooth, quartz grains may produce a degree of roughness		If at all, can only be scratched with difficulty with needle or pocket knife; may contain voids	Acid crystalline rock	Felsite Rhyolite (contains voids) (Quartz porphyry)	These rocks are not easily identified and expert advice may be required

Quartz, Opal	Texture and mineral composition	Colour and lustre	Matrix	Fracture face	Intermixed other components	Additional physical characteristics	Group	Possible rock type	Remarks
Quartz or opal prominent (> 10%, < 50%)	Granular, crystalline	Generally light-coloured; crystals flicker in sunlight	None	Rough; there may be smooth faces of large crystals	None	Elongated crystals may be in parallel or sub-parallel arrangements	Acid crystalline rock	Granite Gneiss (when parallel arrangement of crystals) Pegmatite	Aplite is a fine-grained pegmatite
	Sand grains in very finely grained to dense material	Varying, lustre very dull	Clayey	Rough	Not significant	Scratched easily with needle or pocket knife; feels like blotting paper on wet tongue; gritty feel between teeth when bitten	Argillaceous rock	Mudstone Shale Slate	
					Contains angular to sub-angular fragments of all sorts of minerals and rocks	Strength may vary considerably. Resistance to scratching variable	Diamictite	Greywacke Tillite (Volcanic tuff) (Volcanic breccia)	Recognition of these rocks often difficult, use should be made of expert advice
		White, brown, yellowish-brown, reddish-brown to almost black; lustre generally dull, black parts may flicker in sunlight	Carbonate, iron oxide (red) or iron hydroxide (brown)	Mostly rough, white (carbonate) materials may be rather smooth	Any soil or weathered rock	Scratched easily with needle or pocket knife; white material (carbonate) 'boils' in hydrochloric acid (HCl)	Pedogenic material	Calcrete (white) Ferricrete (other colours)	
Quartz or opal scarce or absent (< 10%; can only be detected by looking carefully)	Very finely 'bedded' with very thin bands of elongated lenses of quartz between often wavy layers of micaceous and other very fine-grained and dense material	Mostly shades of grey, brown or greenish; perpendicular to layers slightly flickering in sunlight; parallel to layers noticeably silky	None	Rough perpendicular to layers; smooth parallel to layers	None	Feels soapy on layer planes which are also scratched easily with needle or pocket knife	Argillaceous rock	Phyllite Sericite schist Slate	
	Granular, crystalline, appears to be composed of one type of mineral only	Mostly reddish or red but may also be white; often contains a few dark green to black minerals (hornblende)		Rough; there may be faces of large crystals		Cannot be scratched with needle or pocket knife but can be with a piece of quartz	Acid crystalline rock	Syenite (Monzonite) (Trachite)	Have determination confirmed by expert because confusion with anorthosite (basic crystalline rock) is possible
		Mostly white but other colours also possible; general flicker in sunlight				Scratched easily with needle or pocket knife; 'boils' in hydrochloric acid (HCl)	Carbonate rock	Marble	
	Granular, crystalline	Generally dark-coloured but a few types are light-coloured or even white. Crystals flicker in sunlight		Rough		Individual minerals react differently to scratching	Basic crystalline rock	Amphibolite Anorthosite (white) Diabase Diorite (light-coloured) Dolerite Dunite Gabbro Norite Peridotite Phonolite	For identification of exact type of rock obtain expert advice

Quartz, Opal	Texture and mineral composition	Colour and lustre	Matrix	Fracture face	Intermixed other components	Additional physical characteristics	Group	Possible rock type	Remarks
Quartz or opal scarce or absent (< 10%: can only be detected by looking carefully)	Granular, crystalline	Shades of green; lustre shiny; flicker in sunlight	None	Variable, tending to be smooth	None	Can be scratched with needle or pocket knife; surface may feel 'soapy'	Basic crystalline rock	Serpentinite	
	Vitreous and dense. May contain empty or filled voids. There may be individual crystals (e.g. olivine) in dense material	Dark shades of various colours; lustre shiny to dull	Dense, uniform mass	Smooth to finely textured, rough	Voids may be filled with opal or even quartz besides other minerals (e.g. zeolites, calcite) which are mostly white to light-coloured	None		Andesite Basalt	Sill phases of diabase or dolerite may be very similar to basalt; if in doubt obtain expert advice
	Very dense and sub-microscopically fine-grained	Colour variable but mostly shades of grey or red; lustre dull	None	Rough	None	Scratched easily with needle or pocket knife; feels like blotting paper on wet tongue; silty or butter-like feel between teeth when bitten	Argillaceous rock	Mudstone Shale Slate	
	Very dense and sub-microscopically fine-grained	Colour often white but also shades of grey and others; lustre dull				Scratched easily with needle or pocket knife; 'boils' in hydrochloric acid (HCl)	Carbonate rock	Dolomite Limestone	Limestone 'boils' much more vigorously than dolomite
				May be rough or smooth	Any weathered rock	Can be scratched with needle or pocket knife; 'boils' in hydrochloric acid (HCl); may contain voids (honeycomb)	Pedogenic material	Calcrete Dolocrete	Calcrete 'boils' much more vigorously than dolocrete
		Colour brown to reddish or yellowish brown, occasionally almost black; lustre generally dull but may flicker in sunlight when black		Rough	None	Nodular structure mostly clearly detectable; broken nodules may contain yellowish, soft, clayey material		Ferricrete	
	Dense to granular	Black, may flicker in sunlight				Very heavy, magnetic (affects needle of compass)	Metalliferous rock	Magnetite	
	Dense to fibrous	White to light green				Surface may feel slightly 'soapy'		Magnesite	Since confusion with other decomposition products of mafic minerals may occur, expert advice is recommended

in the same horizontal division as he proceeds to the right. He will then arrive at the group to which the inspected material belongs.

In the column to the right of that of the group names, individual rock names are shown. They refer to those rocks which have been discussed in Chapter 8. In a few cases, however, names of rocks have been added in parantheses which have not been discussed in this chapter because they have not yet been used for the construction of roads in Southern Africa.

APPENDIX 6: *Table of crushing strength values*

The minimum crushing strength values of natural road construction materials as determined in the 10% FACT or in terms of the ACV and as given in Chapter 8 are listed in Table 33 (p. 217). Most of these values have been specified by the **South African Bureau of Standards** (1976) but some of them are recommendations made in this Chapter. The latter are marked:[1]), and recommendations by **Netterberg** (1971) are marked:[2]).

The first figure in each column is the crushing strength in kN of the dry material and the second figure is the permissible strength of the wet material shown as a percentage of the dry strength. In the case of *argillaceous rocks*, this wet strength is also given in kN.

The Aggregate Crushing Values refer to the dry material only.

The results obtained from the 10% FACT can be converted into the ACV and vice versa by using the expressions (explained in more detail in Chapter 7):

$$ACV = 40 - 0,1083 \ (10\% \ FACT) + 0,000 \ 083 \ (10\% \ FACT)^2,$$
$$10\% \ FACT = 650 - 28,33 \ (ACV) + 0,33 \ (ACV)^2$$
or
$$ACV = 38 - (0,08 \ (10\% \ FACT)),$$
$$10\% \ FACT = 12,5 \ (38 - (ACV)).$$

APPENDIX 7: *Allocation of rocks and soils according to groups of natural road building materials*

Table 34 (pp. 219-222) gives more than 100 names of natural road building materials which may be found in geological and engineering geological reports, in textbooks on geology or petrology, or in explanations of geological maps. These materials are shown against the group of natural road building materials according to which their suitability for road construction should be assessed. The number of names in this list exceeds that given in Table 32, which only contains the names of those materials which are commonly used by Southern African road engineers.

Most of these materials are named according to their mineral composition. They can usually be allocated quite easily to one of the nine groups of natural road building materials, but there are border cases such as rocks composed predominantly of quartz and minerals which change into montmorillonite on decomposition. An example may be quartz-diorite which, in spite of its quartz content, must be classed as a basic crystalline rock because of the relative ease with

TABLE 33
Recommended strength values obtained from crushing tests on the groups of natural road building materials

| Group of natural road building materials | Recommended crushing strength for | | | | Remarks |
| | Surfacing aggregate | | | Base | |
	Rolled-in chips	Surface treatment *)	Bituminous mixtures		
Basic crystalline rocks	10% FACT: 210 kN/75% ACV: 21%	10% FACT: 210 kN/75% ACV: 21%	10% FACT: 160 kN/75% ACV: 25%	10% FACT: 110 kN/75% ACV: 29%	
Acid crystalline rocks	10% FACT: 210 kN/75% ACV: 21%	10% FACT: 210 kN/75% ACV: 21%	10% FACT: 160 kN/75% ACV: 25%	10% FACT: 110 kN/75% ACV: 29%	
High-silica rocks	10% FACT: 210 kN/75% ACV: 21%	10% FACT: 210 kN/75% ACV: 21%	10% FACT: 160 kN/75% ACV: 25%	10% FACT: 110 kN/75% ACV: 29%	The wet 10% FACT on these rocks is of less significance than on other rocks; it might even be neglected
Arenaceous rocks	10% FACT: 210 kN/75% ACV: 21%	10% FACT: 210 kN/75% ACV: 21%	10% FACT: 160 kN/75% ACV: 25%	10% FACT: 140 kN/75% ACV: 27%	110 kN/75% or 29% are only acceptable for *BASE* if cementing matrix siliceous
Argillaceous rocks	unsuitable			10% FACT: 180 kN/125 kN ACV: 24%	*SUBBASE:*[1] 10% FACT: 160 kN/125 kN ACV: 25%
Carbonate rocks	10% FACT: 210 kN/75% ACV: 21%	10% FACT: 210 kN/75% ACV: 21%	10% FACT: 160 kN/75% ACV: 25%	10% FACT: 110 kN/75% ACV: 29%	
Diamictites	10% FACT: 220 kN/70% ACV: 21%	10% FACT: 220 kN/70% ACV: 21%	10% FACT: 170 kN/70% ACV: 24%	10% FACT: 160 kN/70% ACV: 25%	140 kN/70% or 27% acceptable for *BASES* of lightly trafficked roads[1]
Metalliferous rocks	not used			10% FACT: 110 kN/75% ACV: 29%	
Pedogenic materials Calcrete[2])	No information available for Southern Africa		10% FACT: 180 kN/65%	10% FACT: 110 kN/50%? (medium traffic)	*BASES* of lightly trafficked roads perhaps 80 kN/50%?
Ferricrete	Not suitable for crushing tests				
Silcrete	10% FACT: 210 kN/75% ACV: 21%	10% FACT: 210 kN/75% ACV: 21%	10% FACT: 160 kN/75% ACV: 25%	10% FACT: 110 kN/75% ACV: 29%	Compare High-silica rocks

*) It is advisable to consider a slightly greater strength for surface treatment.
[1]) Additions to SABS 1083/1976 (as amended 1979) suggested by H. H. Weinert
[2]) Deviations from SABS 1083/1976 (as amended 1979) suggested by F. Netterberg

which it decomposes and because of the montmorillonite which develops during this process.

The names of a limited number of materials, however, refer to a particular process of transformation of some pre-existing rock or soil and, therefore, their placement in one of the groups often depends on the original material rather than the transformed one. An example of this is mylonite, a rock which has developed from the reconstitution of an extremely intensely broken or crushed rock, i.e. from rock powder, in fault zones of occasionally considerable width especially in orogenic belts.

The following table contains only the name of materials and not those of minerals.

TABLE 34
Allocation of rocks and soils according to groups of natural road building materials

Rock name	Group of natural road building materials
Adamellite	Acid crystalline rocks (related to basic crystalline rocks)
Alnöite	Basic crystalline rocks
Amphibolite	Basic crystalline rocks
Andesite	Basic crystalline rocks
Anorthosite	Basic crystalline rocks
Aplite	Acid crystalline rocks
Arkose	Arenaceous rocks
Banded ironstone	Metalliferous rocks but bordering on high-silica rocks
Basalt	Basic crystalline rocks
Basanite	Basic crystalline rocks
Bauxite	Pedogenic materials
Biotite schist	Arenaceous rocks
Breccia (tectonical)	(depends on broken or crushed original rock)
Calcrete	Pedogenic materials
Camptonite	Basic crystalline rocks
Carbonatite	Carbonate rocks (some may be related to basic crystalline rocks)
Chert	High-silica rocks
Claystone	Argillaceous rocks
Conglomerate	Arenaceous rocks
Dacite	Acid crystalline rocks
Diabase	Basic crystalline rocks
Diorite	Basic crystalline rocks
Dolerite	Basic crystalline rocks
Dolocrete	Pedogenic materials (calcrete)
Dolomite	Carbonate rocks
Dorbank	Pedogenic materials (silcrete ?)
Dunite	Basic crystalline rocks
Essexite	Basic crystalline rocks
Felsite	Acid crystalline rocks
Ferricrete	Pedogenic materials
Gabbro	Basic crystalline rocks
Gneiss	Mostly acid crystalline rocks, but may be basic crystalline rocks
Granite	Acid crystalline rocks
Granite-gneiss	Acid crystalline rocks
Granite-porphyry	Acid crystalline rocks

Rock name	Group of natural road building materials
Granodiorite	Basic crystalline rocks
Granophyre	Acid crystalline rocks
Granulite	Acid crystalline rocks
Graywacke	Diamictites
Greenschist	Basic crystalline rocks
Greywacke	Diamictites
Gritstone	Arenaceous rocks
Harzburgite	Basic crystalline rocks
Hornblendite	Basic crystalline rocks
Hornfels	High-silica rocks
Ijolite	Basic crystalline rocks
Ironstone	Metalliferous rocks
Itabirite	Arenaceous rocks
Itacolumite	Arenaceous rocks
	Basic crystalline rocks
Kersantite	Basic crystalline rocks
Kimberlite	Basic crystalline rocks
Lamprophyre	Basic crystalline rocks
Laterite	Pedogenic materials (compare: ferricrete)
Lava	(variable, more precise denomination required)
Limestone	Carbonate rocks
Magnesite	Metalliferous rocks
Magnetite	Metalliferous rocks
Marble	Carbonate rocks
Melilitite	Basic crystalline rocks
Mica schist	Arenaceous rocks
Migmatite	(variable, but mostly acid crystalline rocks)
Minette	Acid crystalline rocks, may be related to basic crystalline rocks
Monchiquite	Basic crystalline rocks
Monzonite	Acid crystalline rocks (related to basic crystalline rocks)
Mudstone	Argillaceous rocks
Mylonite	(variable, depends on crushed original rock)
Nepheline-basalt	Basic crystalline rocks
Nepheline-syenite	Acid crystalline rocks (related to basic crystalline rocks)
Nephelinite	Basic crystalline rocks
Norite	Basic crystalline rocks

Rock name	Group of road building materials
Obsidian	High-silica rocks, related to acid crystalline rocks
Oceanite	Basic crystalline rocks
Olivine-basalt	Basic crystalline rocks
Oölite	(variable: mostly carbonate rocks, but also metaliferous rocks)
Orthogneiss	(variable: depends on metamorphosed original rock)
Ouklip	Pedogenic materials (ferricrete)
Paragneiss	Acid crystalline rocks
Pegmatite	Acid crystalline rocks
Peridotite	Basic crystalline rocks
Perlite	High-silica rocks (related to acid crystalline rocks)
Phonolite	Basic crystalline rocks
Phoscrete	Pedogenic materials
Phosphorite	Pedogenic materials (phoscrete)
Phyllite	Argillaceous rocks
Phyllonite	(variable, compare mylonite)
Picrite	Basic crystalline rocks
Pitchstone	(mainly high-silica rocks, but influence of many crystalline rocks)
Porphyry	(variable: acid or basic crystalline rocks)
Pumice	Acid crystalline rocks with properties of slag
Pyroxenite	Basic crystalline rocks
Quartz-basalt	Basic crystalline rocks
Quartz-diorite	Basic crystalline rocks
Quartz-gabbro	Basic crystalline rocks
Quartzite	High-silica rocks
Quartz-latite	Acid crystalline rocks (related to basic crystalline rocks)
Quartz-porphyry	Acid crystalline rocks
Rhyolite	Acid crystalline rocks
Sandstone	Arenaceous rocks
Saxonite	Basic crystalline rocks
Sericite schist	Argillaceous rocks
Serpentinite	Basic crystalline rocks
Shale	Argillaceous rocks
Shonkinite	Acid crystalline rocks (related to basic crystalline rocks)
Silcrete	Pedogenic materials
Siltstone	Arenaceous rocks
Slate	Argillaceous rocks
Sparagmite	Diamictites (a type of greywacke)
Spessartite	Basic crystalline rocks
Syenite	Acid crystalline rocks

Rock name	Group of road building materials
Taconite Tephrite Theralite Tillite Tonalite	High-silica rocks Basic crystalline rocks Basic crystalline rocks Diamictites Basic crystalline rocks
Trachite Trachy-andesite Trachy-dolerite	Acid crystalline rocks Basic crystalline rocks (related to acid crystalline rocks) Basic crystalline rocks
Urtite	Basic crystalline rocks
Vein quartz Vogesite Volcanic ash Volcanic breccia Volcanic glass	High-silica rocks Basic crystalline rocks Diamictites with properties of slag Diamictites (variable: acid or basic crystalline rocks)
Volcanic tuff	Diamictites

THE NATURAL GROUND

GENERAL

The term 'natural ground' will be used with reference to the in situ material on which the pavement rests. It is thus fully or largely equivalent to others such as 'subgrade', 'formation' or the 'road bed'. This material is not imported although it may undergo some pre-construction treatment such as compaction or stabilization, or it may be partly removed by scraping, bulldozing or even blasting. If a surface layer, e.g. a clay which is not too thick, is removed entirely to build the road on some stronger material below, the 'natural ground' in this context will be this lower layer upon which the pavement is actually laid. It should be realized that 'natural ground' and 'soil' are not necessarily equivalent. Most natural ground consists of what is colloquially called 'soil' but it may also be outcropping rock.

The properties and the means of selection of rock and soil for use as pavement materials have been described in the previous chapters. Since the pavement also rests on rock and soil however, the natural ground as it affects the overall performance of a road also requires a brief discussion.

ROCK

Although not a road building material in the strict sense, therefore, the natural ground, particularly its bearing capacity, volume change characteristics and permeability, has a marked influence on the design of a road. If the natural ground is rock, hardly any problems will normally be experienced with the bearing capacity, with volume changes and generally with permeability. The high bearing capacity of such natural ground, however, may pose special construction problems and boxing-in will often be required. In Southern Africa, roads are normally constructed on bare rock in deep cuttings which provide boxing-in anyway. Overblasting may be required to improve the permeability and so assist with drainage.

UNSTABLE ROCKS

Rocks, of course should not always be considered as stable natural ground since serious failures may occur in them. Such failures are caused mostly either by the solubility of the rock or by its structural properties.

Sinkholes

Members of the Group of Carbonate Rocks, certain salt deposits and also, in certain areas of Southern Africa, calcrete dissolve in water whose surface effect has

already been considered in Chapter 8. If such rocks are cracked, as is mostly the case, solution occurs along the cracks and fissures into which water penetrates and so widens them in time. The percolating water is almost always charged with carbon dioxide which makes it a weak acid. Loosely packed blown or washed-in soil, rubble derived from the rock itself and possibly insoluble residues fill such cracks without preventing the infiltration and circulation of more water whenever it is available. Where such cracks cross, solution proceeds at a faster rate and cavities develop which grow in size, join up with neighbouring ones and eventually form subsurface caves which are connected with the surface by a number of rubble-filled cracks. There is usually free-flowing water at the bottom of such caves. In most cases, this water represents the water table.

It is obvious from the above that these cavities mostly develop at and just above the water table in those parts of potentially soluble rocks where a network of cracks and fissures allows carbon dioxide-charged surface water to flow more or less freely, to become saturated with dissolved material and to remove these dissolved constituents. It is particularly at this level that the largest underground cavities are likely to develop. Below the water table a certain degree of solution may still take place along cracks following the slow flow of groundwater which gets increasingly carbonate-saturated until it finds an exit at some spring (**Brink**, 1966). Further below the water table, the flow of groundwater is increasingly impeded and the water is eventually so saturated with dissolved material that no more solution occurs. Both the infiltrating and the free-flowing water remove rock and other material in solution or in suspension while the rock, particularly along the cracks and next to their entrance into the cavity, continues to dissolve. Between such entrances, on the walls of the cave and on dry portions of the floor, travertine is deposited in the form of sheets, stalagmites and stalagtites. Eventually the roof of the cavity may become so thin and weak that it collapses and in this way a sink-hole may be formed. Although true roof-collapse, i.e. the break-down of a more or less solid roof, may occur occasionally, the formation of a sink-hole usually begins with the collapse of the loose soil and rubble filling the cracks, in the course of which a still-solid roof portion may be dragged down as well (**Donaldson**, 1963).

Lowering of the water table, which may occur at the beginning of a new erosion cycle, or as it has occurred for instance as a result of the mining activities on the far West Rand, will start a new cycle of cavity formation lower down: older cavities will be increasingly filled with travertine and the stronger downward flow of surface water will have an erosive effect on the filling of the cracks and eventually on any soil cover which may be present. The filling in the cracks will be largely removed and arching may occur in the covering soil until the now-thinning soil cover collapses into the cracks and cavities (**Brink** and **Patridge**, 1965; **Brink**, 1966).

The formation of sinkholes is thus the result of a number of factors:

1) The water-solubility of the country rock; such rocks are the carbonate rocks, calcrete, and salts amongst which gypsum is particularly notorious.

2) The presence of a network of cracks and fissures in soluble rocks which is mostly the case.

3) The availablity of sufficient water to infiltrate and dissolve the rock. This water must leave the rock somewhere, so removing the dissolved material. This makes the occurrence of sinkholes climate-dependent to some extent and most modern collapses in Southern Africa have occurred where N is less than 5. This apparent relation to N less than 5 may of course be something of a coincidence since the rate of suburban development has been faster in these areas than in others with the result that delicate natural equilibria have quite often been disturbed by human activities such as road construction, township development, mining and others which have all caused some disturbance of the pre-existing soil/water equilibrium. This does not exclude, however the possible occurrence of sinkholes in dryer areas where underground cavities may also have formed, perhaps during a past, more humid climate, and may collapse with an increase of human activities.

4) The existence of a general situation in which the flow of the percolating water is sufficiently vigorous to cause the erosion of crack fillings and possibly arching of the overlying soil, and the at least partial removal of loose rubble on the floor of the cavities. This will be the case if the permanent water table has been lowered relatively recently. The position of the water table and the total thickness of soluble rock and soil cover therefore have an influence on the possible total depth and size of sinkholes.

In Southern Africa sinkholes, or at least conditions which may lead to them have so far occurred mostly in the dolomite areas of the western, eastern and northern Transvaal, especially in the area from the vicinity of Pretoria and Johannesburg to the west where the deepest holes and most severe collapses have occurred recently (**Brink** and **Partridge**, 1965; **Brink**, 1966). Minor sinkholes are remarkably frequent in the calcrete deposits of the coastal areas between Bredasdorp and Mossel Bay. The areas with geological conditions suitable for possible sinkhole formation, however, are much more extensive than those mentioned above although hardly anything serious has happened as yet. Most of these areas are in the dry parts of the sub-continent, where N is more than 5 and where development has been rather limited so far.

It is obvious that sinkholes are an extremely severe menace to any type of human activity. The construction of a road and the vibration caused by the traffic may easily trigger a collapse (**Jennings**, 1966) (*Plate 29*). There are no proper remedial or preventive measures to counter this menace and the safest course is to avoid suspect sites or areas. This can only be done through precise site investigations in areas which geophysical surveys have indicated as being endangered. These indications must then be checked and confirmed by means of drilling programmes. In general, areas where the solid rock, e.g. dolomite and/or the normally associated chert, is more than 15 m thick can be regarded as stable for the majority of structures including roads.

It has been pointed out in Chapter 8 that the mode of weathering of carbonate rocks makes them unfavourable for the location of borrow pits for road gravel. The thought may arise, however, that sinkholes, especially older ones where the conditions may have become stabilized to some degree, may be suitable sites for the recovery of road gravel. Such sites are, however, only of limited value because they always involve the considerable risk of further collapse (*Plate 30*) which will endanger the workers and the works and the quantities of loose material contain-

ed in them are usually not very large and are akin to talus deposits consisting of an unsorted mixture of all grain sizes varying from huge blocks to clay-size particles.

Other causes of instability in rocks

Due to the general geological situation of Southern Africa, especially the Republic of South Africa, sinkholes are probably the most severe and regionally the most extensive cause of rock failure. Other conditions may, however, also cause the instability of apparently solid rock. Such conditions are mostly associated with the effects of tectonic movements. It is well known that attention must be given to the gradient of the face of a cutting in relation to the direction of the dip of the strata; the required precautions naturally apply to ordinarily dipping strata as well as to folded strata. Faults and joints may be even more dangerous particularly if they have been overlooked during site investigations because they may cause the collapse of such a face especially in the presence of water (*Plate 31*). Events in the South Western Cape Province during the 1969 earthquake (**Kent,** 1974) showed that the sub-continent is not free from seismic activity and such a possibility must therefore be considered especially in the light of other such events in subsequent years.

In every fault zone, one must expect the occurrence of rocks which have been broken to pieces by the movement of rock masses relative to one another, thus forming a variety of *tectonic breccias*. Such cataclastic structures are called *mylonite* when the extreme condition of pulverized grains has been reached. The latter rocks develop mainly due to overthrust in zones which are tectonically very active, such as orogenic belts, e.g. the Alps. The generally great geological age of cataclastic rocks in Southern Africa has resulted mostly in their reconstitution into solid rocks but cases may occur and have been observed, especially in the South Western Cape Province, where such rocks are critical zones of intense water percolation or may even allow outbreaks of impounded water. All these conditions may lead to rock failures, in most cases to *rockslides* or *landslides*, and it may be noticed that almost all of them are triggered by water. Rockslides and landslides are essentially similar occurrences. In both cases, masses of rock (**Kiekenap,** 1972) or soil (**Legget,** 1962; **Ter-Stepanian,** 1974) slide downhill, the energy for the movement being provided by gravity and the onset of the movement being triggered again mostly by water. The water acts as a lubricant on the plane of sliding, although mass and strength differences could also be the cause of a rockslide on a suitable material, e.g. on a soft shale. Landslides, which usually involve soil rather than rock, differ from *soil creep*, which occurs on almost all slopes, by their spontaneous occurrence but they are not completely unpredictable (**Panet** and **Struillou,** 1970; **Ter-Stepanian,** 1974); maps indicating areas susceptible to landslides could be prepared (**Nilsen** and **Brabb,** 1973; **Řybář,** 1973).

In Southern Africa, landslides have occurred mostly in areas where N is less than 2 and they have usually been connected with a soil layer which is saturated with excessive water. Specially critical sites have been

1) slopes with strong soil creep which can be recognised by the downhill bending of the stems of trees just above their roots or by 'cat steps' – narrow lines of open soil on which the vegetational cover is interrupted and which run closely parallel to the contours (*Plate 32*);

2) older landslides whose newly established equilibrium is disturbed again, e.g. by an eroding stream or by a road cutting.

3) talus slopes.

Such slopes should be avoided when locating the centre line of a road and experience has shown that the neglect of timely warnings may lead, and has led in the past, to costly failures of all types of road.

SOIL

In the above discussion of slides, the inclusion of soils has been unavoidable. Although there are fewer soil types than there are types of rock, the reaction of soils to engineering structures is usually more variable and a separate discussion of soils is therefore justified. Where the natural ground is soil in the colloquial sense, as is usually the case, considerable variations occur and the design of the road must allow for them. The bearing capacity, plasticity and the moisture content and density which can be attained by compaction are those conditions of the soil according to which a pavement must be designed. If a soil is sand or sandy, and if the sand grains are angular, it will be able to carry considerable loads because of the strong interlock between the grains. A sandy soil with angular grains can be compacted more efficiently and its density will be more permanent, although more roller passes may be required, than a soil with rounded grains. If a sandy soil contains some clay, particularly of the kaolinitic type, its bearing capacity is still acceptable and the clay improves its compactiblity. In many cases such soils do not require the provision of a subgrade of selected material, provided they satisfy some minimum requirements regarding CBR and plasticity. Some preparation of the surface will of course always be required. Care must also be taken not to overlook perched water tables which may occur permanently or during wet seasons in the B-horizon of the soil profile and which may have an influence on the durability of the construction materials and the structural design. No general statements can be made, however, about the frequency of occurrence of such conditions since this depends on local conditions. It can only be said that the natural soil will hardly ever meet the requirements for the selected subgrade of a freeway although such soil which is suitable for the subgrade of lightly trafficked roads will often be found, especially in sandy areas.

There are two types of natural ground, with almost opposite properties, which require some special discussion, viz. expansive soils and collapsing soils.

EXPANSIVE SOILS

Wherever clay is encountered in Southern Africa, and more so in the Republic of South Africa than in the territories to the north, it must be assumed to consist of members of the smectite group of clay minerals, montmorillonite in particular, and this assumption must hold until the contrary can be proved. Southern Africa

shares this wide distribution of montmorillonitic clays with only a few other parts of the world, e.g. Ethiopia, the north-western Deccan, parts of Southern Australia, Texas, Israel and parts of the east coast of South America. This is due to the warm-temperature to sub-tropical climates with seasonal rain and the unusually wide occurrence of basic crystalline rocks and argillaceous rocks of late palaeozoic (Karoo) age and younger in these areas.

In Southern Africa, predominantly montmorillonitic clay soils are found on decomposing basic crystalline rocks in areas where N is less than 5. On slopes with a gradient of more than about 5° or in areas where N is less than 2, kaolinite may predominate in the very top layer of the residual soil layer but with montmorillonite lower down. Many of the argillaceous rocks of Karoo age (including those of the Dwyka series) and of Cretaceous and Tertiary age are composed of montmorillonite and montmorillonitic clay soils develop where these rocks weather, regardless of the climatic environment. Kaolinite and other clay minerals predominate in the older argillaceous rocks (**Purnell** and **Netterberg**, 1975).

All clay minerals contain chemically combined water which may be lost or replaced depending on whether the conditions allow drying or wetting. This possible variation in the moisture content makes all clay minerals increase or decrease in volume on wetting or drying respectively. The magnitude of these volume changes varies, however, depending on the type of clay mineral, the quantity of water available and the ease with which absorption or loss of moisture occurs. Clay soils which are composed mostly of montmorillonite are known to be particularly sensitive to such moisture changes: their volume increases considerably when they are wetted and they shrink and crack severely when drying. Under natural conditions these volume changes are controlled mainly by the seasonal variations in rainfall and, to a very great extent, by evapotranspiration especially from certain types of trees and grasses (see Chapter 2). The details of this mechanism have been studied and described by South African authors such as **Brink** (1950a, 1950b), **De Bruijn** (1963, 1975), **Donaldson** (1971), **Jennings** (1961), **Jennings** and **Kerrich** (1962), **Mitchell** (1963) and **Williams** (1965) and numerous others elsewhere, and their findings will be summarised briefly.

The clay minerals of the smectite group, particularly montmorillonite and to some extent also illite, can absorb considerable quantities of water which then becomes chemically combined water. As a result, the minerals increase in volume and so does a soil which is composed of them, i.e. the soil expands. The expansion is mostly directed upward because obviously the soil cannot expand downward and lateral expansion is strongly confined. Consequently, the soil surface moves upward. The total amount of this movement depends on the thickness of the layer of clay and the initial moisture content. Slight sideward movements result in the compression of the clay to such an extent that eventually no more moisture can infiltrate. The amount of upward movement is restricted by the thickness of the overburden and, according to **De Bruijn** (1963), there is no upward movement when the overburden is about 7 to 8 m thick.

When such a soil dries, its volume is reduced and the soil shrinks. Loss of volume in the vertical direction causes the surface level to drop while loss of volume in the horizontal direction causes cracks (*Plate 33*). Such vertical movement is difficult to observe in the open although it can easily be detected by precise levelling. The horizontal movements, however, are very conspicuous: cracking during periods of drying and closure of the cracks when the soil is wetted.

The vertical movement becomes noticeable under covered surfaces where the exchange of moisture by evaporation or evapotranspiration is impeded. Under such surfaces, differential vertical movement is of great importance because of its damaging effects on buildings and surfaced roads. Stresses developed during the movement of expansive clay cause road surfaces to become seriously cracked and concrete slabs to be differentially lifted. Considerable loss of shape may also occur due to the deformation of the lower layers of the pavement. Total failure may be the final result (*Plate 34*).

Gravel roads are as much or little affected by vertical expansion as is the surface of the open veld. However, cracking as a result of horizontal movements, which is so conspicuous in the open veld, is concealed in the layers of a gravel road by the imported gravelly material.

Expansive soils are of course highly plastic and, particularly when wet, their bearing capacity is very low: a soaked field-CBR of as little as 5 may be observed. They are therefore very undesirable foundation soils and their removal and replacement with some more stable material would be the best way to cope with them. This is of course often an uneconomical proposition where roads are concerned because of the often considerable thickness of such soils. The total effect of expansion can be reduced by stabilizing the top 150 mm or more of such a clay with lime thus reducing the plasticity and expansiveness of this part of the soil profile. Many minerals in such soils may be imperfectly structured, there may be relics of primary minerals still in the state of decomposition and there may consequently be varying quantities of amorphous silica. All these conditions may make the soil lime-consuming. It is advisable, therefore, to test the Initial Consumption of Lime (ICL) of such soils prior to lime stabilization (**Clauss** and **Loudon**, 1971).

So far, only clay soils have been considered. It should be understood, however, that there are more types of soil which are liable to swell under suitable conditions and it may perhaps have been better to talk about 'expansive clay' (**Donaldson**, 1971) in the above context. Such other expansive materials are pyrite-containing shales and their weathering residues (**Engineering News Record**, 1960; **Dougherty** and **Barsotti**, 1972; **Grattan-Bellew** and **Eden**, 1975), and soils which are subject to salt migration and crystallization, and frost heave in cold climates. None of these soil conditions has been known to cause problems in Southern Africa so far although the effects of salt migration and crystallization in particular may well be encountered in the future, especially where N is more than 10.

Road construction on expansive soils

There are several methods of coping with the expansive clays in road construction. They will not be discussed here, however, since they are aspects of road design and thus not part of the scope of this book. An additional covering layer, several hundred millimetres thick, of non-expansive imported material reduces the effect of expansion of montmorillonitic or otherwise expansive natural ground to

a certain extent because of the increased load of the overburden and the porosity and compressibility of such a layer. Another possibility is the removal of the top layer of the expansive clay. This is occasionally done to a depth of about one metre, and the clay is replaced with a non-expansive material such as sand. This will not stop the expansion of the clay below but it can be expected that the effects of seasonal rain will be reduced by such an insulating layer which will absorb much of the water before it can reach the clay. The application of this method is of course restricted to short stretches of road because otherwise tremendous masses of soil might have to be moved. Thought has also been given to the reverse of the above method, i.e. keeping the clay permanently wet and expanded by sealing it off from the overlying road structure after thorough wetting (Construction in Southern Africa, 1975). This has been applied in an experimental piece of road which has obviously performed satisfactorily over the last few years.

RHYTHM OF HEAVE

The rhythm of the movement of the soil depends mostly on the seasonal distribution of the rainfall. In a climate where the soil is permanently wet, i.e. where rain falls thoughout the year, such movements do not occur to any noticeable extent and the soil is in a permanently expanded state. Under such conditions, the load of a structure may cause dewatering and settlement until a condition of equilibrium between the load and the climatic environment is obtained. In areas with seasonal rainfall, however, the effect of the expansion and contraction of such soils is very dependent on the season during which the rain falls.

Summer rainfall areas

In summer rainfall areas the soil expands during the warm season. The relatively low evaporation during the dry but cold winter allows desiccation of the soil in the open veld where the soil returns to the condition it was in at the beginning of the preceding rainy season. This is, however, not the case under a covered surface. Under such a surface, there is less desiccation than in the surrounding open veld and the soil therefore does not return entirely to the starting condition. This cycle repeats itself annually until eventually the soil under the covered surface, e.g. a surfaced road, establishes a new moisture equilibrium and noticeable vertical movements are considerably reduced. In the summer rainfall areas of Southern Africa this equilibrium is attained about six to eight years after covering the surface or building the road.

Winter rainfall areas

In the winter rainfall areas, the soil is wetted and expands during the cold season. Evaporation during the dry, hot summer, however, may be sufficiently great to desiccate not only the soil in the open veld but also that under a covered surface if the area is relatively small as under a building or narrow as under a surfaced road. This may result in a continuous upward and downward movement of the surface and a stable equilibrium under such covered surfaces may perhaps never be obtained.

Certain essentially single-sized, very loose soils which contain a limited quantity of a natural binder possess an internal structure which gives them a fairly high bearing capacity as long as their moisture content does not increase. With an increase of moisture and when under load, however, these materials change immediately and spontaneously to a condition which is in equilibrium with the load, e.g. a building or a road, and the new moisture content (Jennings and Knight, 1975). Such sudden settlement which may lead to the severe damage of structures occurs in soils which are known to possess a collapsible grain structure.

Collapse as a result of wetting may occur in all loose materials, including fills and embankments, very dry clays and also highly decomposed basic crystalline rocks (Barden et al, 1973). Most recorded cases of collapse with damaging consequences, however, have occurred in certain types of sandy soil, viz. collapsing sand, collapsing residual granite and collapsing arkose.

Collapsing sand

Collapsing sand has been studied in detail by Knight (1958, 1959). This sand, which Knight considers to be of aeolian origin and to have consisted originally of quartz and other mineral grains, is characterized by the presence of two principal grain sizes, a larger sandy and a smaller more clayey one, while sizes in between are poorly represented. The grading curves have a characteristic S-shape. The sand grains are held together by bridges of iron hydroxide and clay, kaolinite in most cases, derived from the decomposition of the originally admixed mineral grains. These sands possess an open structure, the void ratio is high, and there is very little interlock between the sand grains; the structure is held together by the clay and iron hydroxide bridges. As long as such a sand is not disturbed and is kept dry, these bridges provide considerable bearing strength and their hold on the sand grains is such that the sand will form vertical faces in diggings several metres deep.

If the sand is wetted, however, the bonding bridges between the grains soften and, under pressure above a certain limit, the bridges break and collapse occurs. When this happens, these sands can spontaneously loose up to 20 per cent of their original volume.

Collapsing residual granite

Structurally, collapsing residual granite or rather acid crystalline rocks are very similar to collapsing sand. Again they possess an open, poorly-graded structure, high void ratio and bridges, in this case consisting mostly of kaolinite, which keep the quartz and unweathered feldspar grains together. Again, as long as the material is dry, the bearing capacity is high and the soil will form vertical faces in diggings. If the soil is wetted and the load on it exceeds a certain minimum, it will collapse. These soils have been studied by Brink and Kantey (1961), Knight (1963) and others.

This soil structure is mostly confined to residual acid crystalline rocks whose minerals react differently during decomposition. The most common of such rocks is granite although it is not the only one. Granite is composed of the stable quartz, the slowly decomposing orthoclase and, if present, muscovite, and a few quickly decomposing minerals. The residual soil on such a rock consists of unaltered quartz, usually a quantity of still-unweathered orthoclase, and clay, predominantly kaolinite.

Under topographical conditions which favour easy internal drainage, much of the clay is washed or leached out and the characteristic structure of a collapsing soil develops. The quartz and the unweathered orthoclase form the solid particles which are kept in position by bridges of kaolinite.

The development of such a structure depends on the local topography and climate. Apparently, gradients between 5° and 15° provide the most suitable sites. The ideal conditions for the formation of collapsible structures occur in the lower half of this range in areas where rainfall is relatively high and the rain-water is allowed sufficient time to infiltrate the soil thoroughly, removing all clay from the steeper portion, but only part of it from the gentler portion of the slope. These conditions will be met mostly where N is less than 2. Where N is between 2 and 5, the steeper half of the range from 5° to 15° is more favourable for the formation of the collapsible grain structure because the rain-water has less time to wash or leach out the soil before it evaporates. Therefore, the removal of some of the decomposition products occurs only where faster run-off compensates for the shorter period of effectiveness of the rain-water.

It has been thought that collapsible soil structures derived from acid crystalline rocks are restricted to those parts of the subcontinent which possess an annual water surplus. This is, however, not necessarily true. All that is required is that during some season sufficient rain-water is available to cause that type of internal drainage which allows the partial removal of clay components from the highly decomposed rocks.

Collapsing arkose

Arkose, a member of the group of arenaceous rocks, is a sandstone which contains 25 per cent and more of feldspar, mostly orthoclase. If such a rock weathers where N is less than 5, the orthoclase changes into kaolinite clay. Under favourable environmental conditions, i.e. when topography and climate provide for the removal of part of the clay in the weathered rock, collapsible soil structures develop which are similar to those from acid crystalline rocks.

As already stated, collapse may occur in many more types of soil of different modes of origin. Besides the authors already mentioned, reference may also be made to **Mitchell** and **Van der Merwe** (1958) for, amongst others, the Forest sandstone in Rhodesia or to **Novais-Ferreira** and **Meireles** (1967) for the Angolan 'muceque', a ferruginous sand. **A. A. B. Williams** of the National Building Research Institue, Pretoria, has observed the collapse behaviour of highly decomposed diabase and dolerite at two different sites. Both materials were silty and had a density of only 1 100 kg/m³.
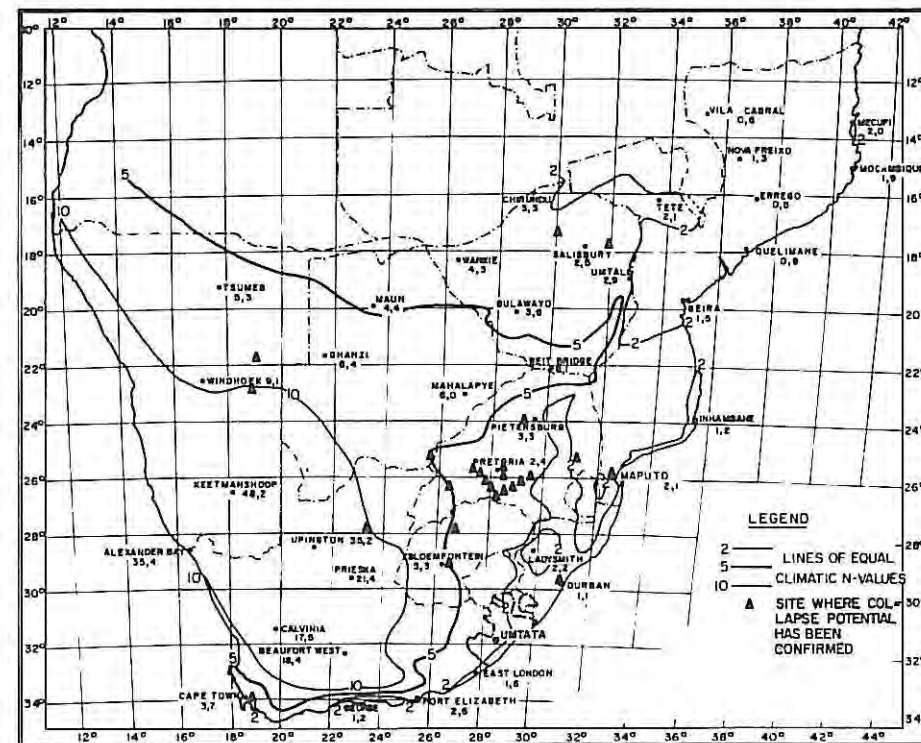
Figure 23: *Sites where the collapse potential of soils has been confirmed*

GEOGRAPHICAL DISTRIBUTION OF COLLAPSING SOILS

Figure 23 indicates that, in Southern Africa, only one case of collapse is so far known in areas where N is less than 2 – collapsing arkose near Durban. It may also be significant that the vast majority of occurrences, all of them either collapsing sand or collapsing granite, have been reported from environments whose N-value is between 2 and 5. Whether this indicates that these climatic conditions are particularly favourable for the formation of collapsing soils cannot yet be proved, although it may be the case. It should be realized, however, that reported collapses have occurred mostly in areas with a high rate of development and that, therefore, the prevalence in areas where N is between 2 and 5 may just be coincidence. Where N is more than 5, collapsing soils are scarce and cases of collapse have not yet been observed where N is more than 10 which does not mean, however, that they are absent in such dry areas.

The load exerted by traffic on a road is sufficient to cause the spontaneous settlement of a soaked collapsible soil particularly in conjunction with the vibrations caused by the traffic (*Plate 35*). This requires the identification of such soil structures in the field during the centre line survey and, when in doubt, in the laboratory. Several methods of identification are available and they have been described by **Knight** (1958), **Knight** and **Dehlen** (1963) and **Jennings** and **Knight** (1975). When undisturbed samples of these soils are inspected with a hand lens the large number of voids that are about the same size as the larger sand grains is apparent. Another means of identification in the field is by comparing the compressibility of a dry and a wet sample of the soil. This can be done by removing two undisturbed, about equally sized samples of the soil, wetting one of them, and squeezing both separately in the palms of the hands to check whether the loss of volume of the wet sample differs noticeably from that of the dry one. **Jennings**, in **Jennings** and **Knight** (1975), has carried this crude identification test further by specifying a definite volume of the two samples, remoulding and reshaping the wet one and definite volume of the two samples, remoulding and reshaping the wet one and thereafter comparing the difference between the volumes. Definite identification is, however, best done by laboratory tests, e.g. the determination of the 'Collapse Potential' (**Jennings** and **Knight**, 1975). It has been suggested that every soil whose dry density is less than 1 600 kg/m^3 must be suspected of being collapsible. In test or auger holes such soils offer little resistance to penetration by the sharp end of a geological pick or they may even crumble easily when scraped with such a pick (**Jennings** *et al*, 1973).

In accordance with the work of **Jennings** and **Knight** (1975), there are a few criteria which can be used easily during field inspections or centre line surveys and which may serve as guides for the recognition of a potentially collapsing soil. The most important prerequisite is probably the partial desiccation of the soil because collapse can only occur in a soil under load when the moisture content of this soil increases. Soils below the water table do not collapse. Such a partly saturated material must also be loosely packed, i.e. its dry density must be low due to the high voids ratio, and the grains must be arranged so that the soil drains freely. In most cases, these grains will be held in position by materials such as clay or iron hydroxide. These natural binders are softened if the soil is wetted and this leads to a rearrangement of the grains resulting in a spontaneous loss of volume of the soil mass. It should be realised, however, that a clean sand can also be made to collapse if the mutual position of the grains allows fast consolidation in the presence of increased moisture. In excavations, collapsing soils appear remarkably uniform and in the desiccated condition they have a hard or stiff consistency.

Collapsible properties are particularly likely to be found in materials or soils such as

1) loose fills,
2) windblown sands that have been subject to weathering for some time,
3) fine colluvium or hillwash of loose consistency.
4) highly decomposed acid crystalline rocks and occasionally even highly decomposed basic crystalline rocks, and

5) certain weathered sedimentary rocks such as arkose but also greywacke.

Most collapsing sands in Southern Africa possess a dark reddish colour which is caused by the high amount of iron hydroxide normally present in the soil. This need not, however, always be the case and the colour may be dirty whitish when iron is absent as is often the case with collapsing granite or arkose. All shades between dark reddish and dirty whitish are actually possible. Therefore, not every red sand is a collapsing sand and the whitish colour of a sand or soil does not exclude the possiblity of collapsible properties.

The mechanism of collapse

The common features of all collapsing soils are then:

a) the presence of loosely packed quartz or other unweathered mineral grains;

b) the presence of weathering residues of clay and iron hydroxides which form bridges between the quartz and other mineral grains;

c) low density due to a high void ratio;

d) high bearing capacity when dry, low bearing capacity when wet, and

e) perhaps the most important, a condition of partial desiccation.

There is a final stage of collapse of such soils, however. This final stage may be attained in steps. The details of this process have been studied by **Jennings** and **Knight** (1975) on whose findings the following description is based.

Let us suppose a loosely packed, cohesionless sandy material which is composed of quartz and other unweathered primary minerals (Figure 24(a)). The engineering properties of such a material are those of many cohesionless sands: no plasticity, satisfactory bearing capacity and easy drainage.

The primary minerals of this sand or soil decompose and change into secondary minerals like kaolinite and limonite (iron hydroxide). The pore water tends to form surface tension blobs on and between the remaining unweathered particles, which are by now virtually all quartz. The secondary minerals, being very small, tend to be attached to these quartz grains or to be compressed between the water blobs, thus forming bridges between the quartz grains (Figure 24 (b)). The typical structure of a collapsing soil has developed.

A collapsing soil as encountered in the field is mostly in equilibrium with the prevailing conditions of load of overburden and field moisture content. If this equilibrium is disturbed, the soil adapts spontaneously to any new condition. Such a new condition could be the application of an additional load like a building or a road. As long as the collapsing soil is kept dry, it can carry a considerable additional load, mostly more than that exerted by a road. If the loaded soil is wetted, however, the clay and other bridges between the quartz grains loose their strength and the soil collapses to the extent required to carry the additional load. This means that such a soil may still retain collapsing properties to some degree and that it would collapse again if a greater load was placed on the wet soil. At some stage, however, the collapse becomes final. This stage is reached when all the voids of a soil are filled with the secondary material (Figure 24 (c)) or, when there is not enough material to fill all the voids, when all the quartz grains touch and support each other.
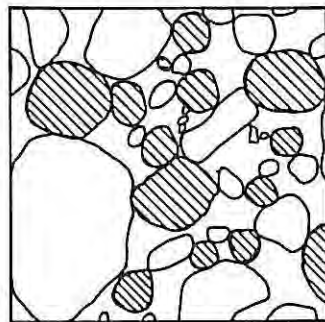
(a)

WIND BLOWN SAND BEFORE
BEING WEATHERED

inferred from (b) below
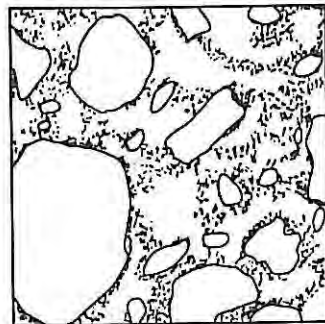
(b)

STRUCTURE OF COLLAPSING
COLL

copied from a photograph
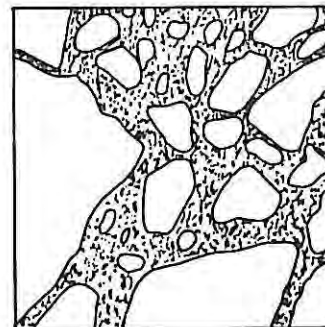
(c)

COLLAPSING SOIL AFTER
WETTING AND CONSOLIDATION

copied from a photograph

LEGEND:

QUARTZ

UNWEATHERED PRIMARY MINERAL

SECONDARY MINERALS

Figure 24: *Schematic representation of the mechanism of collapse of a soil (after Knight, 1959)*

In addition it should be realised that earth tremors and earthquakes may cause a sudden increase in the moisture content of soils. If a soil is structured as described above, this may not only lead to landslides but the soil may also adapt to the changed condition by collapsing (**Seed** and **Silver**, 1972; **Winterkorn** and **Fang**, 1975). This possibility should receive attention and the position of unstable structural elements of the crust, e.g. potentially active faults, should be checked during soil investigations in those parts of the Republic of South Africa (**Kent**, 1974) where seismic activity is known to occur.

Road construction on collapsing soils

In the preparation of collapsing soils for road construction, compaction, possibly to the considerable depth of about two metres, is required. **Mitchell** and **Van der Merwe** (1958) have shown that this can be achieved by vibration rolling after prewetting the soil, or with very heavy pneumatic rollers with low tyre pressure. The achievement of such deep compaction of the soil may pose problems and such rolling is therefore best done during the rainy season. Impact rolling appears to be another suitable method for the deep compaction of such soils as has been shown by **Clegg** and **Berrangé** (1971), **Williams** and **Marais** (1971) and **Clifford** (1976). The advantage of the latter method is that compaction down to a depth of more than one metre can be obtained on dry collapsible soils.

Chemical stabilization of collapsing soils by means of mixing the stabilizing agent into the soil is uneconomical because of the great depth of mixing required. Impregnating the soil with a chemically modified stabilizing agent is a promising prospect which would make deep compaction unnecessary. Such an approach would require, however, that the clay or iron hydroxide bridges should be prevented from losing their strength; this could perhaps be done by controlling the pH of the soil moisture (**Knight** and **Dehlen**, 1963).

It is obvious that expansive clay cannot be used as a road building material. Collapsing soils on the other hand, particularly collapsing sand, is of course suitable for road construction and everything said in Chapter 8 about the use of sands which contain some kaolinite or iron hydroxide applies to collapsing soils as well. Since the property of potential collapse is a consequence of the undisturbed structure of these soils, this property is lost entirely once the soil has been placed in the lower layers of a pavement.

SETTLEMENT

All materials settle when subjected to an additional load and this also applies to expansive soils in which settlement and not expansion occurs when the pressure of the load on a covered surface exceeds the swelling pressure of the clay minerals. In road construction, such loads will of course only occur under high fills.

Settlement and collapse are similar phenomena but settlement is a slow, gradual process in contrast to collapse which is spontaneous. As a result, damage due to settlement can be controlled and timely remedial measures can be taken, but in

collapsing soils preventive measures such as avoiding the inundation of a soil which is already quite heavily loaded, or some means of preconsolidation is required. Once settlement or collapse has occurred, the soil is in equilibrium with the conditions which caused this settlement or collapse and it will remain so unless the load or the moisture content are increased. If the soil has attained the greatest density possible with regard to its grading and the shape and strength of the grains, no further collapse or settlement will occur whatever load is applied.

DELETERIOUS MINERALS

GENERAL

Certain minerals, if present in rock or soil in more than some critical quantity, are disadvantageous or even dangerous if the rock or soil concerned is used as a road construction material. Most of these minerals are not confined to a specific type of rock or even to one of the nine groups of road building materials discussed in Chapter 8, and they and their effect on a road structure have therefore not been discussed with these groups.

The effect of these minerals is of a physical nature in the majority of cases, causing either the deterioration of the aggregate or the disruption of the road structure. Those materials which have a deleterious chemical effect cause undesirable reactions in concrete work and with stabilizing agents.

None of the undesirable minerals has any effect on the road as long as it does not occur on the surface of the stone, i.e. as long as it is not exposed to air, water or potentially reactive construction materials such as cement or lime, or as long as it is not released from the rock by disintegration or in crusher sand. Some of the deleterious minerals are deleterious when fresh and others only in the state of, or after, decomposition.

MICA

Only two of the numerous minerals which form the group of mica occur in such quantities in rocks that are commonly used for road construction that they justify special attention: they are biotite and muscovite. The most striking feature of micas is their excessive platiness. These plates occur singly in sedimentary rocks, in layers in metamorphic rocks, and usually staked in columns in igneous rocks. The platiness and a high elasticity, although varying with the different types of mica, are the main causes of the often disadvantageous behaviour of these minerals in the layers of a pavement and in concrete-making. The disadvantageous properties of mica only take effect of course if the minerals are separated from the rock, i.e. if they occur in weathered rock or soil.

With increasing contents of mica, the liquid and plastic limits of soils increase while the plasticity index decreases. This is caused by the numerous voids contained in a micaceous soil. The high voids ratio makes the soil retain much free water and so the liquid limit increases. The increased internal surface area of micaceous soils, caused by the platy mica minerals, allows a larger quantity of water than usual to be retained in the soil when the cohesion between the soil particles breaks down resulting in an increase of the plastic limit. Since the molecular bonding of the water to the platy minerals results in greater mutual cohesion between these particles than is the case with more bulky minerals, the plastic limit increases at a faster rate than the liquid limit when the percentage of platy minerals increases.

Eventually, when all minerals are platy, the liquid and plastic limits almost coincide (**Tubey**, 1961).

As shown by **Tubey** (1961) and others, these effects are more pronounced in coarse than in fine mica, especially muscovite. **Tubey** defined coarse mica as consisting of plates of more than 0,5 mm in diameter. In soil or weathered rock used for selected subgrade, subbase or even base, the percentage of mica may be allowed to vary to some extent. **Tubey** has stated, however, that weathered rock or soil which contains more than 10 per cent of mica, especially muscovite, particularly if it is coarse, should be avoided for use in any layer of a pavement. No easy and satisfactory method is available for the determination of the percentage of mica in soil and methods which produce reasonable approximations are rather involved. However, the following may serve as a rule of thumb: if mica can be detected at a glance in an engineering soil, the quantity of mica is likely to cause problems, especially if the mica is muscovite.

BIOTITE

Biotite, often descriptively called 'dark' or 'black' mica, is characteristic of igneous rocks and occasionally also of certain metamorphic rocks, such as biotite schist. It decomposes more readily than muscovite and is therefore never a major component of weathered rocks or of soil. Biotite is not as elastic as muscovite and it tends to break more easily into smaller pieces during compaction. No confirmed problems have, therefore, been experienced so far with biotite although the mineral, together with muscovite, has played a part in failures of deep cuts and embankments in Ghana (**De Graft-Johnson** *et al*, 1969). **Müller** (1971), on the other hand, has shown that biotite has little effect on the concrete-making properties of fine aggregate.

MUSCOVITE

Muscovite, the 'light' of 'white' mica, also occurs in igneous rocks, particularly in the acid types like granite and pegmatite, but is also an important and widely occurring component of certain sedimentary and metamorphic rocks. Muscovite is that mica mineral whose road building properties have been studied more often than those of biotite. The reason is probably that muscovite is much more resistant to weathering than other micas and that it therefore predominates in most weathered, mica-containing rocks and in micaceous soils.

Muscovite is an extremely elastic mineral whose platy crystals, if bent, will always tend to recover their original shape. This spring action affects the compactibility of muscovite-containing soils or weathered rocks in which the densities obtained are usually less than 1 770 kg/m^3. In a compacted layer, moreover, muscovite flakes, although rather resilient, may initially have been bent around quartz or other solid soil particles after which they may tend to recover their original shape thus bridging other soil particles and reducing any density obtained initially. **Tubey** and **Bulman** (1964), in their study of highly micaceous silts from Ghana, found that the highest densities were achieved at an optimum moisture content of

20 to 30 per cent less than the plastic limit; an increase of the moisture content by about 7 per cent, however, changed the soil into a wet, almost flowing mass. This reaction of such soils to relatively minor changes in the moisture content, together with the resilience and spring action of the muscovite flakes, is probably the major reason for the difficulties experienced in compacting micaceous soils, especially if the mica is mainly muscovite. This reduction in density occurs in both unstabilized and stabilized micaceous soils. It appears that vibratory compaction is more effective than other methods on such soils. The undesirable effects are aggrevated by the tendency of the muscovite plates to arrange their longer crystal axes, i.e. those defining the prominent crystal planes, perpendicularly to the direction of effort. This means that they all attain an approximately horizontal position in the compacted layer.

It is always advisable to stabilize micaceous soils even if the spring action of muscovite platelets cannot be halted entirely. Cement will be used in most cases to increase strength sufficiently fast to reduce this spring action to some degree. In micaceous clays, lime should of course be used to reduce plasticity, although, because of the slower gain of strength, the density of the layer concerned may decrease slightly. **Tubey** and **Bulman** (1964) have found that satisfactory subbases may be obtained in this manner and that certain micaceous soils may even be used for the bases of lightly trafficked roads.

A study of the influence of mica on concrete by **Müller** (1971) has shown that biotite has little effect on the properties and workability of concrete made with such an aggregate, in contrast to muscovite which profoundly changes the concrete-making properties, particularly those of the fine aggregate. The undesirable effects are presumably again due to the spring action of the flakes which causes a greater water demand in the aggregate probably for the same reasons as those which make the liquid and the plastic limits increase.

Micaceous rocks, e.g. phyllite, mica schist or paragneiss, may give rise to unstable slopes in much the same way as do layers and beds in sedimentary rocks (**Legget**, 1962).

The following materials must always be checked for their muscovite contents:
Acid crystalline rocks: Weathered paragneiss, orthogneiss, granite, pegmatite;
Arenaceous rocks: Weathered mica schist, micaceous sandstone;
Argillaceous rocks: Weathered sericite schist, phyllite;
Transported soils: Muscovite may occur in any of them.

NEPHELINE/ANALCIME

In certain basic magmas which contain excessive potassium, sodium and aluminium in relation to the available silica, the place of feldspars is taken by a group of minerals which are known as feldspathoids, which means something like 'similar to feldspar'. One of these feldspathoids is nepheline, the sodium member of the group which takes the place of the sodium plagioclase albite. Chemically of a compostion between the feldspars and feldspathoids but with added water, are the members of the mineral group of zeolites. Amongst the latter, the cubic mem-

ber analcime – also called analcite – is related to nepheline. Analcime is a zeolite with considerably varying properties and different modes of origin; it can be a primary or secondary mineral. A secondary type is analcime derived from the alteration of nepheline. Under temperature fluctuations, nepheline may change into analcime during which process considerable volume increases occur.

Certain volcanic glasses in basalt or similar rocks may either be of nephelinitic composition or contain submicroscopic nepheline. Many volcanic glasses are unstable and they change into cryptocrystalline minerals, a process called devitrification, over geological times. If a nepheline-containing volcanic rock is exposed to the atmosphere and alternatively heated and cooled by the diurnal variation of temperature, the nepheline component may change into analcime and the associated increase of volume would then make the rock break down into small pieces. This process has become known as 'sunburn' (**Hibsch**, 1938, **Volger**, 1967) and it renders such a rock unsuitable for use as a surfacing stone. However, since this process becomes noticeable only after a year or two, it is not recognized immediately in a quarry face. This particular type of volcanic glass, which occurs only as a subordinate component of the whole rock mass, is also not easily recognized under a microscope. The rock can be recognized as a 'sunburner', however, by boiling it in distilled water for at least 36 hours (**Deutscher Normenausschuss**, 1972): if white spots occur on the surface of the dark to black rock, the process of sunburn is certain (*Plate 36*); if no such white spots occur, sunburn is unlikely although it cannot be discounted entirely.

Sunburn is restricted to the volcanic members of the group of basic crystalline rocks and it is best known in basalt although, theoretically, it may also occur in andesite and phonolite. Sunburn has not yet been observed with certainty in Southern Africa but certain failures of surfacing aggregate in the past may well be ascribed to it.

REACTIVE SILICA

Quartz (SiO_2) has been described as that important rock forming mineral which is most resistant to secondary changes and it has therefore been used as the basis for the classification of natural road building materials given in Chapter 8. This great resistance to, amongst others, weathering, has made quartz the most common mineral in sedimentary and most metamorphic rocks although it is by no means the most common one in igneous rocks.

Quartz is not the only crystallized form, however, in which silica (SiO_2) occurs in nature but there are a number of others which are not as resistant as quartz. Such crystallized forms of silica are the high-temperature modifications tridymite and cristoballite. They may have been left over in igneous rocks as relics of early stages of cooling, and now form a subordinate component in the rock's mineral assemblage. None of these modifications is, however, in proper equilibrium with the surface conditions and over geologically long times they are apt to change into quartz. In this and any other process of rearrangement of the atoms in a silica-containing crystal, a stage occurs when part of the silica is in an amorphous state. In contrast to the high chemical inertness of silica crystallized as quartz, amorphous silica is chemically very reactive.

Amorphous silica is widely spread in nature in a form known as opal; flint, certain cherts and a few other materials, some of them gemstones, are well-known forms of amorphous silica which occur as rocks. In all these rocks and minerals, the silicon and oxygen atoms had no time to arrange themselves in that tetrahedral co-ordination of the composition of SiO_4 which, due to its arrangement in chains where all four oxygen atoms are shared by silicon atoms, results in the average composition of SiO_2. This amorphous silica is also not stable and, again over geologically long times, it changes into some crystallized form. This change occurs in three stages: (a) *amorphous silica*, i.e. opal, (b) *chalcedony*, a fibrous, cryptocrystalline form of quartz, and eventually (c) *quartz*.

Amorphous silica, cristoballite, tridymite, chalcedony and even quartz, the latter if it is intensely fractured or if more than 30 per cent of the quartz crystals in rocks such as granite, gneiss or sandstone are strained due to some degree of metamorphism (**Oberholster** *et al*, 1978), have all become known for their adverse reaction with high-alkali cements, i.e. cements whose alkali content, calculated as $Na_2O + 0,658 K_2O$, is more than 0,6 per cent. The reaction between alkali and these forms of silica leads to the deterioration of concrete through expansion. Since there may be cases, however, where other cements may also react undesirably with the above types of silica, all forms of solid silica, except undisturbed quartz, when used as or accounting for a noticeable percentage of the aggregate, should be considered as potentially reactive. The above modifications of silica may be found in certain acid and basic crystalline rocks, especially their volcanic members which all contain much volcanic glass and tridymite (**Fulton**, 1977), and further in high-silica rocks, siliceous limestone and in arenaceous rocks which have been subject to metamorphism or whose sand grains have been derived from metamorphic rocks. Many of these modifications of solid silica are not readily recognized and the decision on the use of a new concrete aggregate should always be subject to a petrological examination. It is also recommended that established aggregate be examined petrologically from time to time (**South African Bureau of Standards**, 1976).

It may appear somewhat paradoxical to state that pozzolans often contain much amorphous silica and that, notwithstanding what has been said above, their correct admixture is of advantage in concrete-making in several respects, one of which is that pozzolans may counteract alkali reactions. Pozzolans are added as a finely ground powder which forms cementicious materials at an ordinary temperature in reaction with that surplus of calcium hydroxide which often develops during curing especially in fast-setting cements (**Le Sar**, 1977). Pozzolans are obtained from clays and shales after calcination, from opaline materials which occasionally have to be calcined, and from certain volcanic materials and industrial by-products such as blast-furnace slag, fly ash or silica fume (**Blanks**, 1949). It may be of interest to note, as described in Chapter 8, that calcrete containing the siliceous skeletons of diatoms is best stabilized with lime because these skeletons provide the silica required to obtain the cementitious effect. Calcrete which is free of such skeletons must be stabilized with Portland cement.

In general, more care must be taken with reactive silica in cement-rich concrete, e.g. in the construction of bridges, than when working with lean concrete or in soil stabilization. The quantity of cement required for stabilization is much less than that used in structural work and the possibly undesirable effect of reactive silica is consequently also less serious in soil stabilization than in structures like bridges or culverts. Since road engineering is concerned with both, reference should be made to **Fulton** (1977) for details of those aspects of concrete technology which lie outside the scope of the present discussion of natural road construction materials.

SERPENTINE

Another mineral which may affect concrete undesirably is serpentine, a hydrated magnesium silicate. This is an alteration product of olivine and its occurrence is, therefore, restricted to certain members of the group of basic crystalline rocks. The mineral name is serpentine and a rock which is composed almost entirely of serpentine, e.g. an altered peridotite or dunite, is called serpentinite.

Serpentine occurs in various forms one of which is chrysotile which is a type of asbestos and has been used for coarse and fine aggregate in concrete with varying success. For instance, after serpentine had been used successfully for several years, spalling due to large shrinkage movements occurred in a new concrete bridge near Shabani in Rhodesia. This occurrence has been investigated by several researchers and the following general statements can be made:

The water demand and shrinkage of serpentine, when used as a fine aggregate, are very high. When serpentine has to be used, therefore, the aggregate must be screened to eliminate all material smaller than 476 microns, i.e. serpentine must not be used as fine aggregate. Another fine aggregate of good quality must be used instead. For use as a coarse aggregate, only serpentine with the highest strength and the lowest possible chrysotile content should be selected. Serpentinitic coarse aggregate from a stockpile should be presoaked to overcome absorption problems (**Rhodesian Government, Ministry of Roads and Road Traffic**, 1970).

So far, serpentine is not known to have deleterious effects when used in the layers of a pavement.

SULPHIDE MINERALS

A sulphide is a compound of sulphur with often but not always a metal of the general form RS. RS_2 or R_2S. Thus galena, PbS, is a lead sulphide; pyrite, FeS_2, an iron sulphide; and chalcocite, Cu_2S, a sulphide of copper. The deleterious effect which decomposing sulphide minerals have on surfaced roads was first noticed by **Weinert** and **Walker** (1968) but their findings have never been published properly.

Most sulphides are undesirable if they occur in road building materials in more than a critical quantity. Some sulphides occur rather frequently in rocks and have thus obtained a particular importance as contaminants in natural road building materials. They are the iron sulphide FeS_2 with the two modifications pyrite and marcasite, the magnetic pyrite or pyrrhotite, $Fe_{n-1}S_n$ with n ranging from 5 to 16, and the iron-copper sulphide chalcopyrite $CuFeS_2$.

The best known sulphide is the golden iron pyrite, also called 'fools gold', while the silver marcasite and the brown to reddish-brown pyrrhotite are less frequent and less known. Chalcopyrite, also called 'copper pyrite', also has a golden colour but of a darker shade than that of the iron pyrite, often being iridescent purple probably due to the partial decomposition to bornite, Cu_5FeS_4.

If exposed to air and water, these sulphides decompose rather easily and this process is aggravated in the presence of bacteria. Such bacteria are very common and act as catalysts in the process of oxidation of the sulphide. Oxidation is the main feature of decomposition and it normally leads to the development of sulphates and sulphuric acid as shown by the reaction of pyrite in the presence of air and water (**Mrost** and **Lloyd**, 1971):

$$4\,FeS_2 + 15\,O_2 + 2\,H_2O = 2\,Fe_2(SO_4)_3 + 2\,H_2SO_4$$
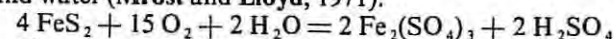
The resulting sulphuric acid (H_2SO_4) is neutralized by bases which are almost always present and more sulphate salts, now not only of iron but also of other metals, such as aluminium, magnesium or sodium, and water are the result.

It is obvious that a road building material which contains components from which a strong acid may develop, may also be a hazard for cement or lime stabilization; however, this does not seem to apply equally to all such minerals within the same rock surface. It may be observed, for instance, that the former position of some pyrites in an exposed rock surface may be indicated only by a rust-coloured filling while other pyrites in the same surface appear to be entirely unaffected. This also applies of course to other minerals, e.g. a clay filling may indicate a former feldspar crystal alongside what appears to be an unweathered one of the same type. These differences in the susceptibility to weathering of similar minerals under the same environmental conditions and within the same rock surface may have various causes, amongst which may be very fine cracks or variations in the number and distribution of unoccupied positions within the crystal lattice. As a consequence, it must be assumed that not all sulphide minerals, amongst which pyrites are the most common, are equally reactive and a simple test has been suggested to distinguish between them: the test sample is immersed in concentrated lime water and within 30 minutes the reactive crystals should produce a brown precipitate while the others remain unaffected (**Fulton**, 1977, p. 414).

The total quantity of pyrite as well as its distribution in the rock may affect the performance of the material concerned. Probably only pyrites which lie on the surface of the rock and are thus exposed to the attack of air, water and possibly bacteria decompose and are the source of sulphuric acid, while those sulphide minerals which are or remain fully embedded in the rock or crushed stone, are likely to have no effect. Witwatersrand quartzites normally contain 0 to 0,5 per cent pyrite but may have as much as 3 per cent (**Fulton**, 1977). The undesirable effect of the decomposition products in the layers of a pavement may become noticeable when sulphide minerals make up more than 1 per cent of the volume of the stone. In this quantity they are detected quite easily with the naked eye or a magnifying glass because of the striking golden or silver flicker when the rock or stone is turned in the

light. The minerals may be distributed evenly throughout the rock or they may occur in clusters with large portions of the rock being free of them. The even distribution appears to aggravate their harmful effect because each piece of stone is likely to contain some of them in its surface and decomposition and the production of sulphuric acid may then occur everywhere in the road layer concerned. If the minerals occur in clusters, this effect will be concentrated in certain spots while other parts of the road layer will not be affected (**Weinert** and **Walker**, 1968). The quantity of sulphide minerals can be determined by point counting on the surface of the stone. Alternatively, the sulphur contents of the material, usually expressed as SO_3, can be easily determined chemically (**Road Research Laboratory**, 1952, pp 96-97, or any textbook on quantitive chemical analyses) but, as already said, if they are recognized easily during visual inspection it must be assumed that their quantity may be a cause of concern.

The risk of encountering free sulphuric acid is likely to increase with the length of the period of stockpiling, especially in areas where N is less than 5. Most sulphide minerals on the surface of the stone will have decomposed after lengthy stockpiling and the former position of the minerals can be recognized by the rusty spots left behind. Removal of the brittle decomposition residue, which consists mainly of salts and iron hydroxide, with a needle helps to detect whether some fresh mineral has been left undecomposed on the surface of the stone. Fresh unaffected sulphide minerals which are left on the surface may be assumed to be of the non-reactive type.

Sulphides can be primary or secondary minerals and they can, consequently, be found in all types of rock. In igneous rocks, mostly in basic and seldom in acid types, the high-temperature modification of pyrite, the mineral pyrrhotite and also ordinary pyrite occur as accessories and only in exceptional cases in harmful quantities. The highest quantity of such minerals is found in rocks into which they have been secondarily impregnated by numerous dykes and veins, and in sedimentary rocks in which they may be part of the deposited material or have been formed by secondary development in conjunction with decaying organic matter. They can therefore be found in large quantities in certain, but not all, arenaceous and argillaceous rocks which have been deposited under water, and in the metamorphic derivatives of such rocks.

In Southern Africa, the undesirable effect of sulphide minerals in road structures has been experienced mostly where natural road building materials have been derived from certain horizons of the Witwatersrand System. Since these materials are always imported for pavement layers, the total quantity of sulphide minerals is limited and the production of sulphuric acid will eventually cease. As long as the material still contains unweathered active sulphide minerals, however, and when it has to be used after some period of stockpiling, the possible presence of free acid must be taken into account when stabilizing such a material with cement or lime. Washing of the material with a solution of high-calcium lime and using a stabilizing agent with enough active lime, will both neutralize the acid to some degree. The results of yet unpublished work by **Netterberg** carried out while this book was being written, suggest that only limited oxidation occurs under most of an impervious surfaced road but that conditions become more favourable for oxidation to-

ward the edges of the surfacing. The consequence of this would be that acid which has formed near these edges or under the shoulders and which might possibly migrate towards the centre line could be neutralized before it becomes harmful if the construction material is treated properly.

It should also be realized that the sulphuric acid which causes trouble probably only to cement- or lime-stabilized materials is not the only deleterious product which develops from these minerals, but that the end-product of decomposition, the sulphate salts, create other more serious problems which are discussed in the subsequent pages.

SOLUBLE SALTS

Systematic research on the effect of soluble salts in Southern African pavements, especially in surfaced roads, only started a number of years after **Weinert** and **Clauss** (1967) published the results of the first study of actual cases of salt failures.

Many minerals are 'salts' which can be dissolved in some suitable medium. 'Soluble salts' with respect to road building materials therefore require a more precise definition: they are salts which dissolve more readily than gypsum in ordinary ground or surface water. There are many such salts but only a few of them may occur in such quantities as to affect a road structure. Such salts are particularly sodium chloride, i.e. ordinary rock salt, and the sulphates of magnesium and sodium, but also the sulphates of iron and aluminium, while calcium sulphate, gypsum, must be regarded as a border case. One carbonate, that of sodium which occurs in nature only as the hydrate $Na_2CO_3.lOH_2O$, Natron, must be added to the list of salts which may cause damage to pavements. Following the above restrictions and limitations, **Netterberg** and **Maton** (1975) have defined 'soluble salts' in this context as follows:

> The term 'soluble salts' is restricted to those minerals which are sufficiently soluble to cause deleterious physical or chemical effects under the conditions normally encountered in civil engineering practice.

Soluble salts are most likely to be encountered in soils of the coastal areas and in arid and semi-arid regions, especially where N is more than 5, or as the decomposition products of sulphide minerals in a number of other rocks. In arid and semi-arid regions they may also be introduced into the construction material by salty compaction water in addition to rising from the subgrade.

If dissolved, salt in the layers of a pavement may be carried upward by moisture. At some level, usually at the contact between the base and the surfacing, the moisture evaporates, especially during the day, and the salts are precipitated. During the night, this process may stop and some of the salt, but not all, may even dissolve again. This process is repeated over many days and nights and more and more salt is precipitated at the contact between base and surfacing. The accumulation of salts start during the construction of the road and it soon becomes noticeable. A white stain on or powdering of the prime, cracks filled with white material in thin surfacings, or surfacing blisters the interiors of which are filled with a white material (*Plate 37*), as well as a whitish layer on the surface of an uncovered base

or unsurfaced shoulder, particularly at the edge of the surfacing, all indicate that salt trouble is imminent. This is particularly the case if the white material also tastes salty. The salts eventually cause the disruption of the bond between surfacing and base or, where blisters have developed, potholing and stripping of the aggregate and loss of density and cohesion of bases, even if stabilized.

Where sulphates are involved, the process may be aggravated by the ease with which these salts hydrate and dehydrate. Some of these salts incorporate a considerable quantity of water in their crystal lattice. Most of this water is expelled again, i.e. the salts dehydrate, at relatively low temperatures. In the case of sodium sulphate (Na_2SO_4) for instance this happens at about 33 °C, and the water is absorbed again below this temperature. The process of hydration and dehydration is associated with volume changes and, since the climate in Southern Africa is largely such that the conditions of temperature and humidity required for the hydration and dehydration of most deleterious sulphates are achieved every day near the contact between base and surfacing, this volume change may in itself affect the strength of the bond between these structural layers.

As important as hydration and dehydration or perhaps even more so, are the forces exerted by crystal growth which have already been discussed in Chapter 4. These forces come into play when the dissolved salts are precipitated and crystallize at the interface between base and surfacing and in the cracks, pores and cavities of the structural layers of the pavement and even within the surfacing. Their magnitude can be sufficient to disrupt even concrete (**Netterberg** *et al*, 1974). The thermal expansion of crystals (**Cooke** and **Smalley**, 1968) may also contribute to the disruption of the top portion of the base and of the contact between base and surfacing as well as the surfacing itself. The effect of thermal expansion must be assumed, however, to be less severe than in the exposed surface of solid rocks, except perhaps in the top portion of a bituminous or concrete surfacing. At the interface between surfacing and base at least, the diurnal variation of temperatures is smaller than that in exposed rock surfaces although it may be higher on the black surface itself.

In general, it seems therefore that a number of forces interact in causing salt damage to roads and that the most effective one appears to be that exerted by crystal growth.

Although salt damage is most severe in surfaced roads, it is not absent in gravel roads. In these roads, most salts will rise to the surface, i.e. to the interface of road and atmosphere, where they will be precipitated, powdered by the traffic and blown or washed off by the weather. Salts which are caught between the gravel of the road structure will of course have an effect due to the same mechanisms as have been described above and this eventually leads to loosening of the gravel, to rutting, potholes and corrugation. The damaging effect of soluble salts in gravel roads is, however, not as severe as it is in surfaced roads and the problem has therefore not caused as much concern in the case of the former as it has in the latter type of road. If atmospheric moisture is available, the most suitable conditions being between rather narrow limits, such salts may act as a sort of 'binder' of the gravel and in this way comparatively smooth and dust-free gravel road wearing courses may be obtained. Such roads require little maintenance. Salt roads have given generally satisfactory performance in certain parts of South West Africa, e.g. in the Walvis Bay-Swakopmund area.

If the atmospheric moisture becomes too little, however, the salts will dry out and rutting, potholing and perhaps also corrugations will result. If the moisture exceeds a certain limit, which may vary from place to place according to other climatic factors, such roads will become slippery, wet and soft.

It would be difficult and probably costly to prevent the upward movement of salts and it would not be possible to keep the salts permanently dehydrated. Therefore, the best remedial measure still appears to be the provision of an impermeable surfacing and possibly the sealing of the shoulders as well. Priming of the base and laying of the impermeable surface seal should be done as soon as possible after completion of the base because this will ensure that the salts remain largely hydrated.

The above remedial, or rather preventive, measures apply to a road which has to be built on salty natural ground, i.e. when the supply of salts is unlimited. When the quantity of salts, either present or developing, is limited, as is the case when its source is imported construction material, e.g. a pyrite containing quartzite, another approach may be considered in addition to the above: after completion and priming of the base, the road is opened to traffic for a limited time, six months or less. During this period, the salts will accumulate between the base and the prime and, when the prime starts breaking up, which will of course happen quite quickly, on the prime as well. The traffic, wind and rain may then assist in the removal of a considerable portion of the salts. After this initial period, the road is completed, preferably along the line set out above, and the limited amount of salts which may still be retained is not likely to cause harm in the future.

Salt-containing materials should preferably be stabilized with agents which contain much active calcium, such as high-calcium lime, even if such an agent has to be added to cement, or by using a larger quantity of stabilizing material than usual for similar, salt-free material. The active calcium may then change certain deleterious salts into gypsum which is less likely to cause problems.

It appears that a road may tolerate more sodium chloride (**Cole** and **Lewis**, 1960; **Smith**, 1962) than most other salts, probably because sodium chloride does not hydrate and volume changes are therefore not involved, as is the case with sulphates. When considering the limiting percentages of soluble salts in a completed pavement, the base in particular, allowance must be made for the upward migration of salts from all layers of the pavement and their concentration at the interface between base and surfacing. Estimates of the limiting percentages have varied over the years, usually tolerating more rock salt, NaCl, than sulphates.

Netterberg (1970), in his preliminary work on the effect of soluble salts in pavements, following on work by **Weinert** and **Clauss** (1967), has suggested that the total very soluble salt content should not exceed 0,2 per cent, determined by a simple electrical conductivity method (**National Institute for Road Research**, 1974). If the total soluble salts content is less than 0,06 per cent, no damage need be expected, no matter what type of salt is present. Caution may be advisable if the percentage is between 0,2 and 0,06 and, for *unstabilized bases* and *subbases*, a conservative approach would be that –

a) a road building material which contains more than 0,2 per cent of soluble salts should be rejected;

b) a road building material which contains less than 0,06 per cent of soluble salts should be accepted, and

c) in road building materials whose percentage of soluble salts is between 0,2 and 0,06, sulphate (expressed as SO_3) should be tested for with barium chloride (BaCl) (**Road Research Laboratory**, 1952; **British Standards Institution**, 1967b (BS1377); **British Ministry of Transport**, 1969). If the test is positive, the sulphate content should be determined and the material should be rejected if this content is more than 0,05 per cent when present as magnesium or sodium sulphates. The analysis with barium chloride should be done on the saturation extract to minimize the effect of gypsum ($CaSO_4.2H_2O$) which only possesses low solubility and is probably not a deleterious soluble salt.

The above values are very conservative and they refer to untreated natural materials. The quantity of 0,2 per cent of very soluble salts is indicated when the electrical conductivity of the saturated soil fines (minus 0,425 mm) is 2,0 mS/cm at 25 °C or 1,5 mS/cm at 25 °C on the minus 6,7 mm fraction determined in accordance with the **National Institute for Road Research** (1974) method CA 21. The quantity of 0,2 per cent should be taken as the limit for the acceptable total of very soluble salts when the above method is applied; however, the use of any other method, such as given in SABS 1083/1976 (**South African Bureau of Standards**, 1976), requires other critical limits. This variation is due to the variation in the solubility of salts and to the different tests which use different quantities of water, allow different lengths of time for the water to take effect and involve other factors so that, as also stated by **Netterberg** and **Maton** (1975), one always has to use the same method of testing which was used to establish the standard.

The work of **Netterberg**, still in progress while this book was being written, further indicates that special precautions are required for the use of salt-containing materials in *bases stabilized with ordinary Portland cement or lime*. Again the maximum electrical conductivity, determined according to the **National Institute for Road Research** (1974) method CA 21, is 2,0 mS/cm or 1,5 mS/cm. In all cases, however, where the conductivity is greater than only 0,4 mS/cm for the -0,425 mm fraction or 0,2 mS/cm for the -6,7 mm fraction the total sulphate content should be determined according to BS1377 (**British Standards Institution**, 1967b). The maximum sulphate content, expressed as SO_3, is 0,25 per cent (**Netterberg**, 1971) if the plasticity index *or* the minus 0,002 mm fraction is greater than 12 per cent before stabilization, and up to 1 per cent SO_3 is permissible if both plasticity and the minus 0,002 mm fraction are less than 12 per cent and if the pH of the saturated soil fines paste is not less than 6 (**Weinert** and **Clauss**, 1967). The salt, sulphate and pH limits of any layer in contact with a cement- or lime-stabilized layer should be similar to those above. If the pH is less than 6 and the conductivity is greater than 2,0 or 1,5 mS/cm, treatment of such material with high-calcium lime until the pH is at least 10,0 may be advantageous. This pretreatment is also advantageous in many cases where the pH of cement- or lime-stabilized materials is below 6.

According to **Blight** *et al* (1974), the tolerable total soluble salt content in sand used for asphalt surfacings is probably 3 per cent. This is probably due to the large number of grains which are fully enveloped by bitumen and thus protected from atmospheric influences. It has been observed that in such sands soluble salt contents which exceed 2 per cent have had no deleterious effect although some of these surfacings turned out to be more susceptible to traffic abrasion than normal.

Coastal sand contains mostly sodium chloride (NaCl), i.e. ordinary or rock salt, as the major harmful contaminant. In dry inland soils where N is more than 5, particularly in solonetzes and solonchaks, chloride and sulphate salts occur and sulphates are often the more harmful salt. The sulphate salts derived from the decomposition of sulphide minerals, as they develop for instance in certain quartzites of the Witwatersrand System, may of course be found wherever these rocks occur or are dumped.

GYPSUM

It has already been said that the calcium sulphate, gypsum, occupies a borderline position in the consideration of soluble salts. Gypsum is very much less soluble than the other salts whose solubility has so far been considered as the major cause of their deleterious effect on road structures. Gypsum is the hydrated form of calcium sulphate, $CaSO_4.2H_2O$, and the anhydrous state is anhydrite, $CaSO_4$. Both occur as minerals, evaporites, in nature. The chemical notations show that the mineral absorbs less water during hydration than do most of the soluble salts discussed so far and consequently the associated volume changes are also smaller.

Gypsum is not known to cause problems in pavements in the form of blistering of the surfacing or other types of deterioration as experienced with other soluble salts. Where it has been said that gypsum has been responsible for such problems, as e.g. in parts of South West Africa, closer inspection has revealed that other salts, especially rock salt (NaCl), appeared to be at the root of the trouble (**Netterberg**, 1970). Gypsum has been used for the bases of roads in the north-eastern Sahara in a very dry climate (N presumably more than 20) under a lightly sanded, viscous, cutback tack coat. The material has given satisfactory service. These bases contained 30 to 95 per cent of finely divided gypsum (30 to 60 per cent minus 0,08 mm) but coarser gypsum was not used (**Baudet** *et al*, 1959). No reason was given for the rejection of coarse gypsum; it should be realised, however, that coarse calcium sulphate is very likely to be anhydrite and that volumetric changes due to hydration and a consequent change into gypsum may occur even in a very dry climate after the rather weak anhydrite has been crushed to a fine material during compaction. None of the above bases was stabilized.

The stabilization of gypsum-containing materials may well cause problems if the gypsum content is above a certain limit which may be as little as 0,5 per cent. Failures due to expansion and cracking following on sulphate attack may then occur.

ADDITIONAL PROBLEMS WITH CONCRETE

Concrete plays a role in road construction mainly in bridges, culverts and rigid surfacings. So far in the discussion of deleterious minerals much reference has already been made to their reaction with cement. Cement, as distinct from organic materials such as bitumen or tar, reacts directly with a number of the aggregates, changing them or being changed itself. Most of these reactions are required to attain the desired goal of increased strength but some prevent this goal from being attained. Excessive shrinkage or expansion are the most usual undesirable effects and they lead to cracking of the concrete.

The effect of gypsum on cement- or lime-stabilized bases has already been mentioned. Amongst the sulphates which commonly occur in nature, gypsum is the least deleterious one, magnesium sulphate is the worst and sodium sulphate is intermediate (**Fulton**, 1977). The effect of the rarer iron and aluminium sulphates is probably comparable to that of magnesium sulphate and, in addition, they may acidify the aggregate. The rather rare barium sulphate (BaSO$_4$) does not cause problems. In the choice of the aggregate and the correct cement, it is therefore important to consider not only the sulphate ion concentration but also the metal ions.

Besides the minerals already mentioned, virtually all decomposing primary minerals may cause shrinkage or expansion of concrete to a lesser or greater degree and the freshness of such minerals must therefore be ascertained during the selection of concrete aggregate. The mafic minerals, e.g. hornblende, pyroxene or olivine, are more sensitive than the felsic ones, e.g. orthoclase. It is obvious that the clay minerals themselves are even more deleterious than the decomposing primary minerals. A considerable number of South African aggregates, especially from rocks of the Karoo System, have been considered to be 'shrinking aggregates' (**Stutterheim**, 1954; **Roper** et al, 1964) and engineers have been very hesitant to use them for many years. More recent work and experience have shown, however, that the occurrence of such shrinking aggregates is not as widespread and the problem not as serious as was previously thought (**Davis**, 1975). In general, there are many more minerals which are deleterious to concrete than those which are harmful in a road structure. The minerals which require special precaution in concrete-making have been recorded and described by **Fulton** (1977) and the standards required for concrete aggregate from natural sources have been specified by the **South African Bureau of Standards** (1976).

MINE SAND

Although mine sands are not minerals, some remarks on them may be appropriate, particularly on those obtained from the sand and slime dumps in the Witwatersrand area.

These materials consist of very finely ground quartzite and they have been used with varying success for slurrying or as fine aggregate for other purposes. In many cases, the performance of the material was as sound as could be expected from quartzite. In other cases, however, cement stabilization proved inefficient. It was noted that problems only occurred when the 'yellow mine sand' was used while none were experienced with the 'white mine sand'.

The difference between the two sands appears to be that the 'white mine sand' is a waste product obtained after the introduction of the cyanide method of gold processing, while the 'yellow mine sand' dates back to the times when gold processing still involved sulphuric acid. Furthermore, the very small grain-size of these materials will also mean that almost all the pyrite has been separated. Pyrite may have been quite abundant in the original rock and may have subsequently decomposed, producing free sulphuric acid and iron oxides. A portion of the sulphuric acid from either source may still be present in these materials and in certain cases it may even have contaminated underlying dumps of coarser material, as e.g. that obtained during shaft-sinking.

The effect which yellow mine sand has on a road is almost the same as that caused by rocks containing sulphide minerals in which free acid has developed. Prior to use, therefore, the pH of such a sand must be determined and if it falls in the acid range, more precisely if it is below 6 (**Weinert** and **Clauss**, 1967), the sand must be treated like aggregate containing sulphide minerals; the same precautions apply. If the road has to be built over such a dump or over any other mine waste, the dump must be considered as a natural ground with an almost inexhaustible supply of acid and salts and the only protective measure appears to be porous, easily draining, selected subgrade which does not provide for capillary upward movements of moisture from the sand to the layers of the pavement, and possibly an impervious surfacing.

Generally speaking, the older dumps consist of sand and the later ones of the more finely ground 'slimes'. When free of acid or sulphates or when stabilized so that sufficient reactive calcium was available, the sands have been used on the Witwatersrand with success up to selected subgrade and the slimes for fill.

APPENDIX 8

APPENDIX 8: *Special properties of the principal natural road construction materials*

In this Appendix, the natural road construction materials have been tabulated alphabetically under the names commonly used by the Southern African road engineers. The tables contain properties of individual rocks which could not be included in the general description of the properties of the various groups of natural road building materials in Chapter 8 but which have been discussed in other parts of this book, especially in Chapter 10. This Appendix is meant to draw the attention of the reader to such special properties and to assist him in the location of the relevant discussion.

Rock name	Group of natural road construction materials	Special properties in addition to those described under the relevant group	Remarks
Amphibolite	Basic crystalline rocks	None	
Andesite	Basic crystalline rocks	May contain amygdales filled with amorphous silica (see Chapter 10)	
Anorthosite	Basic crystalline rocks	None	
Aplite	Acid crystalline rocks	None	Only used in Rhodesia where it is a fine-grained *pegmatite*
Arkose	Arenaceous rocks	None	Feldspars in this rock are orthoclase which does not cause problems
Basalt	Basic crystalline rocks	Nepheline/analcime content may cause 'sunburn' (see Chapter 10). Amygdales may be filled with amorphous silica (see Chapter 10)	
Calcrete	Pedogenic materials	None	
Chert	High-silica rocks	Chalcedony and cryptocrystalline quartz may exhibit properties typical for reactive silica (see Chapter 10)	
Conglomerate	Arenaceous rocks	May contain pebbles which are liable to decompose (see Chapter 4); may occasionally contain pyrite (see Chapter 10)	
Diabase	Basic crystalline rocks	None	
Diorite	Basic crystalline rocks	None	May occasionally contain some quartz
Dolomite	Carbonate rocks	None	
Felsite	Acid crystalline rocks	None	May polish (see Chapters 6 and 8)
Ferricrete	Pedogenic materials		Sandy host soil better than clayey host soil
Gabbro	Basic crystalline rocks	None	

Rock name	Group of natural road construction materials	Special properties in addition to those described under the relevant group	Remarks
Gneiss	Acid crystalline rocks	May contain excessive muscovite (see Chapter 10)	This refers equally to ortho- and paragneiss
Granite	Acid crystalline rocks	May contain muscovite and strained quartz (see Chapter 10)	
Greenschist	Basic crystalline rocks	May occasionally contain serpentine (see Chapter 10)	
Gritstone	Arenaceous rocks	None	
Hornfels	High-silica rocks	May contain amorphous silica (see Chapter 10)	
Ironstone	Metalliferous rocks	Certain types (especially the 'banded ironstone') may contain amorphous silica (see Chapter 10)	
Limestone	Carbonate rocks	None	
Magnesite	Metalliferous rocks	None	
Magnetite	Metalliferous rocks	None	
Marble	Carbonate rocks	None	
Mica schist	Arenaceous rocks	Contains much mica and precautions are required if the mica is muscovite (see Chapter 10)	
Mudstone	Argillaceous rocks	None	Inclined to disintegrate rapidly if exposed to the atmosphere
Norite	Basic crystalline rocks	None	
Pegmatite	Acid crystalline rocks	May contain large crystals of muscovite (see Chapter 10)	Compare *aplite*
Peridotite	Basic crystalline rocks	May contain serpentine (see Chapter 10)	
Phonolite	Basic crystalline rocks	Could show signs of 'sunburn' due to presence of analcime (see Chapter 10)	
Phoscrete	Pedogenic materials	None	

Rock name	Group of natural road construction materials	Special properties in addition to those described under the relevant group	Remarks
Phyllite	Argillaceous rocks	None	Likely to cause very unstable slopes, e.g. in cuttings
Quartzite	High-silica rocks	Check contents of sulphide minerals, especially pyrite (see Chapter 10)	'Witwatersrand' quartzite is a mixed material consisting of numerous different rocks (see Chapter 8)
Rhyolite	Acid crystalline rocks	Often rich in volcanic glass (amorphous silica) (see Chapter 10) and often high voids ratio	
Sandstone	Arenaceous rocks	Check for muscovite and sulphide minerals (pyrites) (see Chapter 10)	
Sericite schist	Argillaceous rocks	None	Likely to cause very unstable slopes, e.g. in cuttings
Serpentinite	Basic crystalline rocks	Not recommended as aggregate in concrete (see Chapter 10)	
Shale	Argillaceous rocks	May contain sulphide minerals (pyrite) (see Chapter 10)	
Silcrete	Pedogenic materials	Contains much amorphous silica (see Chapter 10)	
Slate	Argillaceous rocks	May contain sulphide minerals (pyrite) (see Chapter 10)	
Syenite	Acid crystalline rocks	None	Likely to polish more than other acid crystalline rocks (see Chapters 6 and 8)
Tillite	Diamictites	None	
Vein quartz	High-silica rocks	Occasionally with galena (PbS, lead sulphide) which has not yet become known to cause problems	

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INDEX

INTRODUCTION

The alphabetical arrangement of this index is based on the major terms which all start with a capital letter. Subtitles of these major terms are indented and arranged alphabetically under the major term concerned. In a number of cases, sub-subtitles had to be introduced as well and they are further indented and also arranged alphabetically under the relevant subtitle. The alphabetical arrangement of subtitles and sub-subtitles is done according to their central term, neglecting auxiliary words such as 'of', 'in', 'in relation to', 'causing' and others. Every higher order of terms continues after all lower-order terms have been listed.

References to page numbers of the text are made in **ordinary print**, those to tables only in **bold** and those to figures only in **parentheses**.

Where several consecutive pages are given against one index item, e.g. '73-76', one of these page numbers may be bold or in parentheses. However, where the page numbers at the beginning and end of a sequence refer to either a table (**bold**) or a figure (**parentheses**) the text pages in between are shown separately, e.g. **65**, 66, **67** and not **65-67**.

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